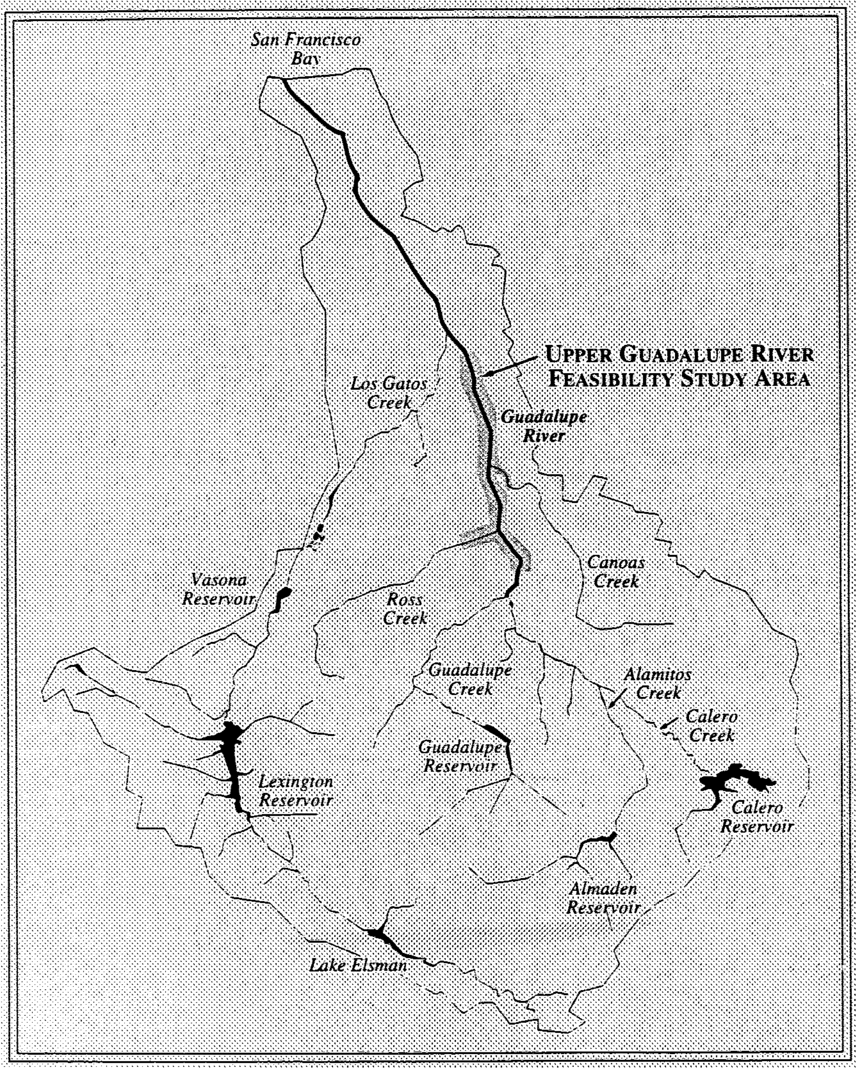
UPPER GUADALUPE RIVER
FINAL FEASIBILITY REPORT

TECHNICAL APPENDICES

Upper Guadalupe River Feasibility Study



U.S. Army Corps of Engineers
San Francisco District

And
Santa Clara Valley Water District

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Santa Clara Valley Water District
And
U.S. Army Corps of Engineers, San Francisco District

UPPER GUADALUPE RIVER FINAL FEASIBILITY REPORT

VOLUME II

TECHNICAL APPENDICES

JANUARY 1998

UPPER GUADALUPE RIVER - APPENDICES

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PART I

SUMMARY OF HISTORICAL
PLAN FORMULATION PROCESS

SUMMARY OF HISTORICAL PLAN FORMULATION PROCESS

Description of Preliminary Flood Protection Alternatives

Preliminary flood protection alternatives for the upper Guadalupe River were formulated during two successive studies: the 1985 Corps of Engineers Guadalupe River Interim Feasibility Report, and the 1989 Corps of Engineers Upper Guadalupe River Reconnaissance Study. Each of these preliminary plans are described below. The 1985 study eventually led to the construction of a flood control project downstream of the study area in downtown San Jose. The remainder of this summary pertains to previous formulation of alternatives within the study area of the upper Guadalupe River.

Nonstructural Measures

A number of nonstructural management measures were evaluated and are summarized in the following paragraphs .

Flood Forecast, Warning and Evacuation. This measure would provide a system for predicting the occurrence of hazardous floods and warning to the affected populace to evacuate the flood-prone area. The measure would reduce risk of injuries and loss of life. Reduction of flood damages would be limited to structure contents which are portable and automobiles which can be driven out of the floodplain. Structures, property, yards, streets and public works would still be inundated. The cost of a flood warning system would be relatively low. Its effectiveness, however, would be limited because of the small size of the watershed and the relatively short duration of flooding. There is enormous difficulty in accurately predicting the floods and there is uncertainty that the residents would be readily available to evacuate damageable property contents upon such short notice.

Temporary or Permanent Closures of Structure Openings. This measure would implement temporary or permanent closure of openings to structures in flood prone areas. For those windows and doors which cannot be permanently closed, temporary flood shields would be fabricated to be placed over first-floor openings when flood events are forecast. This measure must be coordinated with the Flood Forecast, Warning and Evacuation measure described above in order to be effective. Damageable contents would be protected during flood events, but structure exteriors, yards, driveways, streets and public works would still be inundated. Most of the existing residential and small commercial buildings in the study area are wood frame and in the event of large floods would not withstand the hydrostatic uplift pressures. However, much of the flooding is from shallow sheet flow and it is also very difficult to make the woodframe construction impermeable even to shallow flooding. Therefore, the measure was not considered to be physically feasible and was eliminated from further consideration.

Raising Existing Structures. This measure would reduce flood damages to individual structures and contents by raising the structure above the floodplain. The measure is physically feasible for wood-frame structures on raised foundations, but problematic for most other structures. Although the structures and contents would be protected, property yards, driveways, streets and public works would still be flooded. Many of the existing structures in the study area have slab-on-grade foundations which makes the alternative prohibitively expensive. The measure is also not physically feasible for larger residential and commercial structures. The cost of raising structures in place depends on the height to which the structure is raised. The cost was expected to be approximately 25 to 50 percent of a structure's value. More than 8,000 structures are located within the 500-year floodplain. Raising of a limited number of structures may be feasible but most of the floodplains are relatively large since once the floodwaters leave the channel they do not return because the channel is perched. The measure was eliminated from further consideration due to the costs associated with raising the large number of structures in the floodplain.

Small Walls and Levees Around Existing Structures. This measure would reduce flood damages to individual structures and damageable contents by encircling individual structures with floodwalls or levees. In order to allow access to the protected structures, openings would be required for driveways and doors with temporary closures fabricated and placed across the access openings. If temporary closures are used, this measure would require coordination with the Flood Forecast, Warning and Evacuation measure described above. This measure would not prevent inundation of streets, driveways and other access points.

Rearranging or Protecting Damageable Property Within a Structure (Floodproofing). This measure consists of rearranging or protecting at least a portion of valuable, damageable property contents which can be moved from one area of the structure to another. Since the most economical method of implementation is to move high-cost items from lower floors to upper floors, this measure is often only practical for multi-story buildings. Because only some items are relocated, residual damage to structures, contents that are not relocated, property yards, streets and public works would still occur. This nonstructural plan involves rearranging movable, damageable contents within flood prone structures. Rearranging or protecting damageable property contents would be limited to items which could actually be moved. The reduction in flood damage would offset the costs of rearranging contents within structures, but additional business costs would incur from relocating inventories to less accessible areas.

Purchase or Removal of Existing Structures and/or Contents From Floodplain. This measure would involve acquiring properties or removing structures in the floodplain. Flood damage reduction would be limited to the structures and damageable contents which are removed from the floodplain. Residual damages to remaining structures, property yards, streets and public works would still occur. This measure would not be economically feasible due to the large number of structures involved. It would result in a significant loss of tax revenue. Removing homes would also cause significant social impacts. Residual damages to remaining structures, streets and public works would cause service disruptions and would also result in cleanup costs. For these reasons, this measure was eliminated from further consideration.

Flood Insurance. Federally subsidized flood insurance is provided through the National Flood Insurance Program (NFIP), which is administered by the Federal Emergency Management Agency. The NFIP has already been implemented in the study area. Approximately 7500 structures are located in the upper Guadalupe River 100-year floodplain. Approximately 50 percent of the owners (nearly 3800 structures) are currently purchasing flood insurance. Property owners may still experience financial losses associated with damages which are not covered under their policies. This measure has been incorporated as part of the future without-project conditions.

Structural Measures

Upstream Reservoirs. Two upstream reservoir measures for reducing flood damages were previously studied by the Corps of Engineers and by the Santa Clara Valley Water District. The concept of these measures was to construct flood control reservoirs upstream on Guadalupe Creek and Alamitos Creek in order to reduce peak flow rates through the storage of floodwaters and thus eliminating the need to enlarge the channel capacity of the Guadalupe River. However, these measures were dropped from consideration because of the large costs, environmental impacts and effectiveness in controlling the runoff in the study area.

Channel Modification. Throughout the previous Corps of Engineers studies, six basic channel modification features have been considered. The features included four variations of channelization and two floodwall variations. These measures are briefly described below.

1. Earth Bypass Channel: This channel modification feature increases the capacity of the river by constructing a parallel earthen bypass channel which may remain dry during low flows, but carries excess flows during flood events. During floods, the natural channel carries all flows up to its existing capacity, and excess water in the channel is diverted into the earth bypass channel which would carry the floodwaters further downstream to a point where the river channel is capable of carrying the flow. This measure allows the natural channel to remain untouched, except at the diversion and reentry points, and creates additional open space which can be used for recreational or other compatible uses. A large right-of-way outside of the natural channel is required, which typically involves relocation of homes and businesses in urban areas.

2. Covered Bypass Channel: This measure is similar in concept to the earth bypass channel, but rather than excavating a surface channel, a reinforced concrete box culvert would be buried beneath the existing ground surface. The land above the bypass could be used for streets, parking areas, or open space. This measure preserves the natural channel and requires less right-of-way than an earth bypass, but involves higher construction costs.

3. Widened Earth Channel: This measure would require excavation along one side of the natural channel only. The channel capacity would be designed to achieve flow velocities which are slow enough to prevent channel erosion. No rock or concrete lining would be required on the channel bottom or side slopes. The excavated bank would be planted with native grasses, shrubs and trees. The opposite bank of the channel would be preserved in its natural condition. The measure requires a slightly wider right-of-way than a lined channel.

4. Widened Rock Channel: This measure would require excavation of the natural channel on one side only, but would have slightly narrower channel dimensions than the Widened Earth Channel. As channel width decreases, water velocities increase. In order to prevent damaging erosion associated with higher velocities, the narrower channel would require that the channel bank be lined with rock. This measure requires less right-of-way than the unlined channel, but the natural condition of the channel along the excavated bank is lost.

5. Floodwalls: This measure consists of reinforced concrete floodwalls which are built parallel to the tops of both river channel banks. Floodwalls are only practical where the natural channel has a relatively large carrying capacity, so that the floodwall height is not so high as to be unacceptable. The natural channel would be preserved, but homeowners along the riverbank may view the floodwalls as an intrusion and the floodwalls may adversely impact local interior drainage systems.

6. Floodwalls with Selective Clearing: This measure is similar to Measure 5 above except that brush and low shrubs along the channel banks would be cleared. Clearing the channel banks increases the channel capacity by reducing flow resistance in the river. Thus, lower floodwalls can be used. This reduces the construction costs and aesthetic impacts. However, the natural condition of the channel would be disturbed.

1989 RECONNAISSANCE STUDY

A reconnaissance study, completed in 1989, reformulated the preliminary plans in order to address flooding in the upper Guadalupe River reaches. The reconnaissance study resulted in carrying forward two structural alternatives, the Widened Earth Channel Plan and the Earth Bypass and Widened Earth Channel Plan, for further consideration during this feasibility phase of the study. The Widened Earth Channel plan was determined as the most efficient alternative for providing flood control protection. The No Action plan was also carried forward as a baseline condition against which to measure the impacts of the two structural plans. The reformulated plans are discussed in the following section.

No Action

The No Action plan is the "without-project" condition that serves as the basis for developing and comparing action oriented plans. Under the No Action plan, it is assumed that no Federal or local project would be constructed to reduce the flood hazard from the upper Guadalupe River.

Widened Earth Channel Plan

This plan would widen the east bank of the existing channel between Highway 280 and the confluence with Canoas Creek. The existing streambed would remain undisturbed. In order to alleviate induced flooding along Canoas Creek, 5-foot high floodwalls would be constructed along both banks of Canoas Creek from its mouth to the Capitol Expressway crossing. Excavation of the east bank of the river would resume near Berkshire Drive and continue upstream to Branham Lane. Three areas of west bank widening would also be required. Project features are summarized in Table 1.

Table 1 - Widened Earth Channel Plan Features

IMPROVEMENT	DOWNSTREAM LIMIT	UPSTREAM LIMIT
Channel Widening - east bank	Highway 280	Canoas Creek
Channel Widening - west bank	Hillsdale Avenue	Capitol Expressway
Floodwalls	Canoas Creek at confluence	Canoas Creek and Capitol Expressway
Channel Widening - west bank	300 ft above Capitol Expressway	500-ft upstream
Channel Widening - west bank	1200 ft below Ross Creek	300 ft above Ross Creek
Channel Widening - east bank	Berkshire Drive	Branham Lane

Bypass and Widened Earth Channel Plan

This plan is identical to the Widened Channel Plan described above, with the exception of the addition of an earthen bypass between Highway 280 and Willow Glen Road. The major features of the plan are summarized in Table 2 below.

Table 2 - Earth Bypass and Widened Earth Channel Plan Features

IMPROVEMENT	DOWNSTREAM LIMIT	UPSTREAM LIMIT
Bypass Channel	Highway 280	Willow Glen Way
Channel Widening - west bank	Hillsdale Avenue	Capitol Expressway
Floodwalls	Canoas Creek mouth	Canoas Creek and Capitol Expressway
Channel Widening - west bank	300 ft above Capitol Expressway	500-ft upstream
Channel Widening - west bank	1200 ft below Ross Creek	300 ft above Ross Creek
Channel Widening - east bank	Berkshire Drive	Branham Lane

Comparison of Preliminary 1989 Reconnaissance Plans

Table 3 presents a comparison of the two reconnaissance study plans. Both of these plans were carried forward in the current feasibility study, but were reformulated into the candidate plans described in the following section.

TABLE 3

1989 Recon Study

SUMMARY COMPARISON OF ALTERNATIVE PLANS

		Widened Earth Channel		Earth Bypass Channel	
PLAN DESCRIPTION	**NO ACTION**	**PLAN A**	**PLAN A**	**PLAN B**	**PLAN B**
Level of Protection	N/A	50-Year	100-year	50-year	100-Year
Excavation	N/A	300,000 CY	962,000 CY	282,000 CY	868,000 CY
Land Purchase	N/A	21.7 acres	29.3 acres	28.1 acres	37.4 acres
Relocation	N/A	Replace 4 vehicle & 2 railroad bridges	Replace 6 vehicle & 2 railroad bridges	Replace 4 vehicle & 2 railroad bridges	Replace 6 vehicle & 2 railroad bridges
Floodwall	N/A	Canoas Cr. (3-4 ft. high for ±2 miles) Ross Cr. (± 2 ft. high for ±1/2 mile)	Canoas Cr. (4-5 ft. high for ±2 miles) Ross Cr. (± 4 ft. high for ±1/2 mile)	Canoas Cr. (3-4 ft. high for ±2 miles) Ross Cr. (± 2 ft. high for ±1/2 mile)	Canoas Cr. (4-5 ft. high for ±2 miles) Ross Cr. (± 4 ft. high for ±1/2 mile)
1. NATIONAL ECONOMIC DEVELOPMENT					
Construction Costs					
Project First Cost	N/A	$47,288,000	$71,602,000	$52,019,000	$75,888,000
Federal First Cost	N/A	$23,644,000	N/A**	$26,010,000	N/A**
Non-Federal First Cost	N/A	$23,644,000*	N/A**	$26,010,000*	N/A**
Interest During Const.	N/A	$ 3,868,000	$ 6,333,000	$ 3,733,000	$ 6,099,000
Project Cost+Interest	N/A	$49,405,000	$77,935,000	$53,194,000	$81,987,000
Annual Costs					
Interest and Amort. (100 years, 8.875%)	N/A	$ 4,541,000	$ 6,570,000	$ 4,721,000	$ 6,900,000
Project O & M Costs	N/A	$ 277,000	$ 435,000	$ 267,000	$ 420,000
Total Annual Cost	N/A	$ 4,818,000	$ 7,005,000	$ 4,988,000	$ 7,320,000
Offsetting Land and Structure Resale Cost	N/A	$ 374,000	$ 431,000	$ 399,000	$ 439,000
Annual Benefits					
Flood Damage Reduction	N/A	$ 3,916,000	$ 4,528,000	$ 3,916,000	$ 4,528,000
Other Benefits	N/A	$ 1,414,000	$ 1,695,000	$ 1,414,000	$ 1,695,000
Total Annual Benefits	N/A	$ 5,330,000	$ 6,223,000	$ 5,330,000	$ 6,223,000
Total Net Benefits	N/A	$ 886,000	($717,000)	$ 497,000	($1,052,000)
Benefit-to-Cost Ratio	N/A	1.20 to 1	0.90 to 1	1.10 to 1	0.86 to 1

*Limited by 50% of project first cost.

**Cost sharing not applicable on unjustified alternatives.

TABLE 3

(Continued)

1989 Recon Study

SUMMARY COMPARISON OF ALTERNATIVE PLANS

		Widened Earth Channel		Earth Bypass Channel	
	NO ACTION	PLAN A	PLAN A	PLAN B	PLAN B
		50-year	100-year	50-year	100-Year
2. ENVIRONMENTAL QUALITY ACCOUNT					
Air Quality	Vehicle emissions increase.	Temporary impact, construction equipment emissions.	Temporary impact, construction equipment emissions.	Temporary impact, construction equipment emissions.	Temporary impact, construction equipment emissions.
Water Quality	Improved by local programs.	Temporary temperature increases.	Temporary temperature increases.	Temporary temperature increases.	Temporary temperature increases.
Noise	No impact	Construction noise	Construction noise. Remove noise buffer u/s of Capitol Exp.	Construction noise.	Construction noise. Remove noise buffer u/s of Capitol Exp.
Riparian Veg.	Maint. continues to impact vegetation.	Significant loss of vegetation: ±20 ac. to be removed.	Most significant vegetation loss: ±24 ac. removed.	Least disruptive of all plans: ±14 ac. to be removed.	Significant loss of vegetation: ±18 ac. to be removed.
Fish and Wildlife	Habitat impacts to continue.	Significant loss of habitat.	Greatest loss of habitat of all plans.	Least disruptive to habitat of all plans.	Significant loss, but less than Plan A, 100-year.
Endangered Species	Maint. continues to impact candidates.	No endangered; candidates may be affected.	No endangered; candidates may be affected.	No endangered; candidates may be affected.	No endangered; candidates may be affected.
Wild and Scenic Rivers	N/A	N/A	N/A	N/A	N/A

TABLE 3
(Continued)

1989 Recon Study

SUMMARY COMPARISON OF ALTERNATIVE PLANS

		Widened Earth Channel		Earth Bypass Channel	
	NO ACTION	PLAN A 50-Year	PLAN A 100-year	PLAN B 50-year	PLAN B 100-Year
Cultural	Possible damage to archaeological & historical sites.	No National Register sites affected.	No National Register sites affected.	No National Register sites. affected.	No National Register sites affected.
3. REGIONAL ECONOMIC DEVELOPMENT					
Local Govt. Finance	No change from existing con-	Local sponsor share of plan would be $2.3 million in cash.	N/A**	Local sponsor share of plan would $2.6 million in cash.	N/A**
Economic Development	No effect	Temp. increase in construction jobs. Reduced economic losses due to flood protection.	Temp. increase in construction jobs. Reduced economic losses due to flood protection.	Temp. increase in construction jobs. Reduced economic losses due to flood protection.	Temp. increase in construction jobs. Reduced economic losses due to flood protection.
4. OTHER SOCIAL EFFECTS					
Recreation	No change from existing plans	Effect would be beneficial as trail system is accommodated.	Effect would be beneficial as trail system is accommodated.	Effect would be beneficial as trail system is accommodated.	Effect would be beneficial as trail system is accommodated.
Aesthetics	Slight change due to maintenance clearing	Moderate impact on aesthetic quality.	Greatest loss of scenic values.	Least impact on aesthetic quality.	Less degradation of scenic values than Plan A, 100-year.

**Cost sharing not applicable on unjustified alternatives.

TABLE 3

(Continued)

1989 Recon Study

SUMMARY COMPARISON OF ALTERNATIVE PLANS

		Widened Earth Channel		Earth Bypass Channel	
	NO ACTION	PLAN A 50-Year	PLAN A 100-year	PLAN B 50-year	PLAN B 100-Year
Transportation	Future highway construction to be built.	Traffic conditions improve due to new bridges.	Improved traffic conditions due to new bridges.	Improved traffic conditions due to new bridges.	Improved traffic conditions due to new bridges.
5. RESPONSE TO ASSOCIATED EVALUATION CRITERIA					
Acceptability	N/A	Not acceptable to local sponsor; does not provide 100-year flood protection.	Not acceptable to local sponsor; Plan B, 100-year preferred.	Not acceptable to local sponsor; does not provide 100-year flood protection.	Acceptable to local sponsor. Bypass Channel is locallly preferred plan.
Completeness	N/A	Complete	Complete	Complete	Complete
Effectiveness	N/A	Effective in providing 50-year protection.	Effective in providing 100-year protection.	Effective in providing 50-year protection.	Effective in providing 100-year protection.
Efficiency	N/A	Most efficient of all plans: Maximimizes net benefits.	Not efficient: costs exceed benefits.	Next most efficient of all plans: fewer net benefits than Plan A, 50-year.	Not efficient: costs exceed benefits.

HTRW

Hazardous, Toxic and Radiological Waste

A project area review and preliminary site assessment (PSA) of the study area was conducted in two phases by Kleinfelder, Inc., as described in their January and August 1992 reports. The purpose of the studies was to (1) identify potential sources of surface and subsurface contamination, (2) evaluate potential impacts of existing contamination sources, (3) identify potential impacts of contamination, and (4) provide recommendations for additional investigation, as necessary to evaluate the extent and impacts of contamination to the project design and construction.

The PSA identified twenty-four properties, located between Interstate Highway 280 and Blossom Hill Road, as having a high potential for contamination impact. A high potential means that the properties are known or suspected to be contaminated, based on the following criteria:

1. Existing or former presence of underground or aboveground storage tanks;
2. Storage and use of hazardous materials, including agricultural pesticides; and
3. Site located adjacent to a property with known contamination.

Subsequent to the PSA, the Santa Clara Valley Water District conducted a Level II HTRW investigation, which was documented in April 1995. The results of the Level II investigation indicate that there are several areas of concern along the project alignment. These areas are described below, summarized in Table 1.

(1) Two small areas of stained soil on parcel APN-264-48-005, Bennett's Automotive, are impacted with elevated concentrations of mercury. A total of 20 cubic yards of impacted soil should be excavated and removed prior to property acquisition.

(2) Approximately 89 cubic yards of soil below a storm drain outflow pipe and near the west gate on parcel APN 434-04-002. This soil may contain elevated concentrations of chlordane and DDT and dinoseb. In addition, this soil also contains petroleum hydrocarbon concentrations. Removal of this soil from the property is recommended prior to property acquisition. If the soil removal is not completed prior to construction, the soil will have to be excavated and properly disposed of during construction activities.

(3) Petroleum hydrocarbons have impacted soil and groundwater at a site at the corner of Willow Street and Lelong Avenue. The hydrocarbon plume appears to have impacted at least three parcels (APN 434-04-006, -011 and -066). The estimated volume of impacted soil is 16,400 cubic yards.

(4) Parcel 434-20-023 (Elks Lodge) was investigated as part of the bypass channel/island bank assessment. The upper 5 feet of soil at the Elks Lodge site appears to contain mercury concentrations at elevated levels. Either additional characterization should be

performed to show that the soil is not hazardous with respect to mercury, or the upper 5 feet of soil should be excavated and properly disposed.

(5) The Valley View Packing Facility (APN 455-12-007, 39-003, 39-007) is the site of a documented fuel release and some pesticide impact in shallow soil. The volume of pesticide impacted soil is estimated to be 4,720 cubic yards. The volume of petroleum hydrocarbon impacted soil is estimated to be 5,000 cubic yards.

(6) Soil on parcels 458-17-001, -006, -017 and 458-16-027 were proposed for use as wetland construction material. Nickel and silver exceeded the guidelines for cover and non-cover soil. Mercury and pesticides exceed the guidelines for cover soils but not non-cover soils.

Table 1: Potential HTRW Sites

Assessors Number	Address	Contaminants of Concern
264-48-005	384 McLellan Ave	Mercury
434-04-002	Undeveloped	Chlordane, DDT, Dinoseb
434-04-006	450 Willow St	Petroleum Hydrocarbons
434-04-011	1127 Lelong St	Petroleum Hydrocarbons
434-04-066	456-458 Willow St	Petroleum Hydrocarbons
434-20-023	Elks Lodge	Mercury in top 5' of soil
455-12-007, 455-39-003, & 455-39-007	Valley View Packing Plant	Petroleum Hydrocarbons
458-16-027, 458-17-001, 458-17-006, & 458-17-017	Vacant	Nickel and Silver

MITIGATION

UPPER GUADALUPE RIVER FEASIBILITY STUDY

INCREMENTAL MITIGATION ANALYSIS

Bill DeJager, Planning Branch
August 20, 1997

1.0 Introduction

The purpose of this analysis is to compare the cost-effectiveness of different mitigation options and determine the optimum mitigation plan for cost-sharing purposes. The information in this analysis is based upon information in the U.S. Fish and Wildlife Service (FWS) Revised Draft Coordination Act Report (CAR) (FWS 1997) and Corps of Engineers planning guidance (Army Corps of Engineers 1995).

This analysis considers mitigation for the Bypass Channel Plan. This plan, including its mitigation features, was devised by the local sponsor for this study, the Santa Clara Valley Water District (SCVWD). The issue at hand is how much of this mitigation plan should be cost-shared by the Corps of Engineers.

2.0 Planning Environment

2.1 Habitat Types

Flood control improvements along the upper Guadalupe River would affect several habitat types. The FWS has rated these habitats according to their importance for fish and wildlife and their recommendations regarding the importance of avoiding and mitigating losses of these habitat types (FWS, 1997). Habitats of the highest importance are categorized as Resource Category 1: no loss of existing habitat value. No habitats in the study area are in this category. Habitats of high value but which are capable of being replaced in a reasonable time period are categorized as Resource Category 2: no net loss of in-kind habitat value. Habitats of somewhat lesser value are categorized as Resource Category 3: no net loss of habitat value, while minimizing loss of in-kind habitat value. No habitats in the study area are in this category. Habitats of low value are placed in Resource Category 4: minimize loss of habitat value.

The primary terrestrial habitat of concern in the study area is riparian forest, in Resource Category 2. Riparian scrub-shrub is also placed in Resource Category 2 (although the FWS had placed it in Resource Category 3 until very recently in the planning process). Impacts upon both of these habitat types were analyzed in a Habitat Evaluation Procedure (HEP) study. Riparian shrub-shrub was evaluated using riparian forest evaluation species as it is simply an early seral stage of riparian forest. Although this habitat type has been classified as Resource Category 2 (no net loss of in-kind habitat value recommended) , the FWS has indicated that mitigation for losses of riparian forest can be applied against losses of this habitat type.

Another habitat type of concern is urban forest, which consists of concentrations of trees in residential areas that provide some songbird habitat. The FWS has placed this habitat in Resource Category 4; however, the Bypass Channel mitigation plan includes replacement of this habitat on a 1:1 basis. All other terrestrial habitat types were placed in Resource Category 4 by the FWS.

Several wetland and aquatic habitat types are also of concern. Shaded riverine aquatic (SRA) cover is the nearshore aquatic zone which occurs where the adjacent riverbank supports riparian vegetation that overhangs the water. This cover type is particularly important for salmonids (salmon and trout) due to its moderating effect on water temperatures, but has many other habitat benefits as well. Similar in character is shaded palustrine aquatic (SPA) cover, which occurs along the margins of the in-stream percolation ponds in reach 12. This cover type does not particularly benefit salmonids due to its seasonality and other factors, but provides other habitat benefits. Both of these cover types were classified as Resource Category 2.

Finally, freshwater emergent marsh, which occurs in certain portions of the river bed, has a high value for wildlife. The FWS has classified this habitat type as Resource Category 2.

2.2 The Bypass Channel Mitigation Plan

Construction of the Bypass Channel Plan and associated mitigation areas would remove substantial amounts of riparian forest, riparian scrub-shrub, urban forest, ruderal herbaceous vegetation, and SRA cover. Smaller amounts of upland landscaping would also be removed. Low-quality SPA cover along the margins of the percolation ponds in reach 12 would be removed due to channel widening in much of that reach.

This mitigation plan was devised by the SCVWD with input from the California Department of Fish and Game (CDFG) and the FWS. This plan proposes replacement of lost riparian forest at a 2:1 ratio, lost urban forest habitat and freshwater marsh at a 1:1 ratio, and replacement of SRA cover on a 1:1 basis as measured in linear feet. All habitat types would be replaced in-kind, except for urban forest which would be replaced with a native oak-sycamore forest which would probably be of higher habitat value. The compensation ratios used for this plan were determined in consultation with resource agencies (although they are not necessarily endorsed by the FWS) and are based in part on earlier HEP work done by the local sponsor.

This plan also proposes creation of SRA cover along Guadalupe Creek upstream of the study area through utilization of biotechnical methods. This mitigation measure was not evaluated in the HEP, was not included in the Corps study, and will not be discussed further here. Also, SPA mitigation was not addressed in the HEP study (as the FWS gave no indication until very recently that this cover type had particular value), but this cover type would be substantially increased and enhanced in quality due to riparian forest plantings in reach 12.

The key attributes of this plan are as follows. First, all the elements of this plan were preformulated and screened for effectiveness by the local sponsor. Second, all elements of this plan are independent and additive, with the exception that SRA cover mitigation is a intentional and inseparable consequence of placing riparian forest mitigation adjacent to the river.

2.3 Economic Considerations Affecting Mitigation

The proposed project would be located within a major urban area with very high land costs, commonly $350,000 to $650,000 per acre. Mitigation areas to be located on project lands would incur relatively low costs as land costs would be assignable to project construction rather than mitigation (as described below). Installation costs are estimated at $70,000 per acre for riparian forest, $60,000 per acre for urban forest, and $25,000 per acre for freshwater marsh (except for areas requiring grading).

All mitigation areas proposed under the Bypass Channel Plan would be within the channel boundary as defined by the Sacramento District's Real Estate Division (CESPK-RE) for purposes of their gross appraisal, with the exception of proposed mitigation areas in reach 13, which is just outside of the project boundary. CESPK-RE has determined that a non-standard estate will be utilized for mitigation areas within the project boundary. Since the easement for the floodway will be extremely restrictive, additional rights will not be needed for mitigation areas, so these areas will not be acquired in fee. Thus, mitigation areas within the project boundary will have no real estate costs associated with them.

Mitigation areas outside of the project boundary would incur very high real estate costs except if located within the river channel. With unit real estate costs being so high, real estate will be the major factor determining mitigation costs and cost-effectiveness. The one exception is proposed mitigation within the channel in reach 13, which has an estimated real estate cost of $1,000 per acre. However, only limited amounts of mitigation land are available in this reach. Channel lands farther upstream are considered by the local sponsor to not be available for mitigation use, reach 6 (just downstream of the study area) is preempted by flood-control improvements and mitigation planned by the local sponsor, and lands downstream of that reach are preempted by the downtown Guadalupe River project.

Therefore, mitigation outside of the project boundary would generally be excessively expensive ($420,000-720,000 per acre for combined land and installation costs). Conversely, giving the social and regulatory importance of fish and wildlife habitat in the area, failure to utilize available project land for mitigation (given its nominal real estate cost) would constitute a missed opportunity.

2.4 Social and Regulatory Considerations

Riparian forest and freshwater marsh are three habitat types which have severely declined in Santa Clara County and the State of California during the past century. Estimates of statewide losses range as high as 90 to 95 percent. Associated SRA cover, fish, and wildlife have declined substantially as a result. The Chinook salmon which utilize the Guadalupe River may not be indigenous to this river, but have suffered such a regional decline that one run (Sacramento River winter run) is now listed as endangered and others are under consideration for listing. The steelhead trout, which spawns in the river, is currently proposed for listing as threatened effective October 1997.

The resource agencies (U.S. Fish and Wildlife Service, National Marine Fisheries Service, and the California Department of Fish and Game) have indicated a high degree of concern regarding project impacts to terrestrial and aquatic habitats. The impending listing of the steelhead trout as threatened or endangered provides additional impetus to these concerns as well as stricter legal requirements for the minimization of short-term impacts and avoidance of net long-term impacts. The cumulative impacts of this project and the downtown Guadalupe River project (given the severe, and as yet unmitigated, SRA cover impacts of the downtown project) heighten these concerns.

Within the Santa Clara Valley, natural habitats are scarce and are an important amenity. Local citizens and environmental groups have made it clear that maintenance of local environmental quality and minimization of impacts on vegetation, fish, and wildlife is very important.

Given these regulatory and social constraints, full mitigation of project impacts appears to be imperative to the success of this project. Additional habitat restoration beyond that minimally needed to cover project impacts would be desirable if it can be accomplished in an economically efficient manner.

2.5 Biological Considerations

Riparian forest mitigation is a relatively straightforward matter compared to some other types of mitigation. This habitat type grows rapidly and is easy to establish if site conditions are suitable and adequate attention is paid to ensuring the initial establishment of plantings. Existing riparian forest in the impact areas ranges from very young to somewhat mature, and its habitat values can be replaced during the lifetime of the project. One caveat is that riparian forest mitigation plantings should be located adjacent to or at least close to aquatic habitat to ensure that the mitigation is in-kind. True in-kind mitigation for riparian scrub-shrub habitat is not needed, as riparian forest mitigation will provide the same sort of habitat values (plus additional values).

One issue worth noting is the question of equal compensation versus in-kind compensation of habitat values. Five avian species were used in the riparian forest HEP. Under equal compensation, the objective is for the total number of habitat units lost due to project construction to be replaced, regardless of which HEP species these habitat units accrue to. In other words, tradeoffs are allowed between HEP species. Under in-kind compensation, habitat units lost by one species may not be replaced by habitat units accruing to another species. In the HEP team meeting on November 18, 1996 it was agreed by most parties present that at least limited tradeoffs should be allowed between species.

However, the revised draft CAR includes a policy determination by the FWS that in-kind compensation should be used to determine mitigation needs. Regardless, the Corps will use equal compensation because none of the species is considered to be any more important than the others, and equal compensation provides a more accurate assessment of how much mitigation is needed to compensate for losses of overall habitat value.

SRA cover is created by the juxtaposition of riparian forest and riverine habitat. It can also be recreated during the lifetime of the project by locating riparian forest mitigation plantings adjacent to the river. Biotechnical erosion control work proposed by the local sponsor also will create some SRA habitat value, although to a lesser degree.

3.0 Mitigation Increments

As all elements of the local sponsor's mitigation plan have been preformulated and were screened by the local sponsor for effectiveness, so the incremental mitigation analysis will consist only of the aggregation of these plan elements into increments and an analysis of the economic efficiency of these increments.

The HEP study only evaluated riparian habitat, consisting of riparian forest and riparian scrub-shrub, and aquatic habitat as enhanced by SRA cover and SPA cover. The latter cover types are created by the existence of riparian forest adjacent to riverine and percolation pond aquatic habitats, respectively. Terrestrial habitats in resource category 4 (urban forest, ruderal herbaceous, and urban landscaping), as well as freshwater marsh, were not included in the HEP. This limits the extent to which mitigation increments for different habitat types can be compared. However, tradeoffs of habitat units between important habitat types (such as riparian forest and freshwater marsh) would not be acceptable to the resource agencies.

3.1 Riparian Forest/SRA Cover

The HEP did not evaluate the number of habitat units that would be created by different management actions. Rather, due to high land costs in the area, it was assumed that mitigation would be conducted on an intensive basis. Given the contrast between real estate costs as high as $650,000 per acre and installation costs mostly $70,000 per acre or less, this is a valid approach (note however, that installation of the proposed riparian forest/SRA cover mitigation in reach 10b and wetland mitigation in reach 12 would be substantially more expensive due to major grading expenses). Also, the HEP did not evaluate the relative effectiveness of riparian forest plantings in different locations in producing habitat units. Therefore, all riparian forest mitigation plantings will be assumed to be equally effective in producing habitat units for riparian forest species, and only mitigation costs will be compared across different increments.

As discussed above, for lands within the project boundary there are no real estate costs assignable to mitigation. Therefore, all riparian forest mitigation up to and including full mitigation of project impacts will constitute one increment. Within this increment there is no basis for differentiating either costs or benefits of mitigation.

However, additional riparian forest mitigation can be divided into several increments. It should be noted that while this mitigation is not needed to compensate for riparian forest losses, some of this mitigation is needed for compensation for SRA cover losses not fully covered by the first increment. The second increment includes additional mitigation at the same cost level as the first increment. The third increment includes all riparian forest mitigation in reach 13, which has minor real estate costs. The fourth increment consists of riparian forest mitigation in reach 10b; this mitigation has substantial grading expenses. The fifth and final increment will consist of an additional two acres of riparian forest mitigation on currently vacant land in reach 12, with high real estate costs. These increments are compared in Table 1.

Table 1: Riparian Forest Mitigation Increments

ID	Description	Acreage	Cost	Cost per Acre
R-1	On-site/needed/inexpensive	14.60	$1,022,000	$70,000
R-2	On-site/extra*/inexpensive	4.03	282,100	70,000
R-3	Off-site (reach 13)/extra*/inexpensive	1.29	91,590	71,000
R-4	On site (reach 10b)/extra*/expensive	2.52	726,400	288,254
R-5	Off-site (reach 12)/extra/expensive	2.00	840,000	420,000

*While these mitigation increments provide extra riparian forest habitat value beyond that needed to achieve full mitigation under equal compensation, these areas are still needed to mitigate for losses of SRA cover.

3.2 Additional Mitigation for SRA Cover Losses

The proposed riparian forest mitigation would fall about 11 percent short in mitigating losses of SRA cover, according to the aquatic HEP. However, this HEP will be analyzed and critiqued by a consultant and this shortfall may decrease as a result. The local sponsor has

proposed off-site mitigation for SRA cover losses, to be located on Guadalupe Creek upstream from the study area. This off-site mitigation was not analyzed in the HEP.

Additional mitigation measures to improve conditions for salmonids have been proposed by the local sponsor. These measures were not analyzed in the HEP process. The first measure is construction of vortex rock weirs at intervals along the river. These serve a dual function as grade control structures and as aquatic habitat enhancements, at no extra cost relative to standard grade-control structures. The second measure is the removal of existing partial barriers to fish passage, and the third measure is construction of a fish ladder at the mouth of Ross Creek. The costs of these latter two measures are not readily separable from other project costs so will not be considered further here.

3.3 Urban Forest

As the HEP did not assign habitat units to urban forest, mitigation of this habitat can not be compared to riparian forest mitigation using the standard methodology. However, a qualitative comparison of cost-effectiveness is still possible. Table 2 shows urban forest mitigation. No differentiation of costs or benefits could the done within this category due to the absence of assigned real estate costs (for reasons discussed above) and the lack of any basis for assigning a higher habitat value to one parcel or another. Total mitigation is 0.05 acres greater than the habitat loss. Note that per-acre costs are quite reasonable compared to the riparian forest mitigation increments.

Table 2: Urban Forest Mitigation Increment

ID	Description	Acreage	Cost	Cost per Acre
U-1	Urban forest	3.61	$216,600	$60,000

3.4 Freshwater Marsh

The Bypass Channel mitigation plan includes creation of 2.42 acres of freshwater marsh. Of this amount, 1.05 acres is to compensate for habitat losses within the Corps study area. An additional 0.80 acres is to compensate for habitat losses downstream due to locally-funded improvements (reaches A and 6), and 0.57 acres constitutes extra mitigation. The Corps should not pay for the cost of mitigating for local improvements, so only the other mitigation quantities will be considered here. Habitat values for freshwater marsh were not assessed in the HEP process. However, this habitat type is known to generally have a very high value for wildlife.

The proposed freshwater marsh mitigation can be broken up into several increments based upon cost, as shown in Table 3. No HEP data are available to differentiate between the habitat benefits of the increments, so only mitigation costs will be considered in the formulation of increments.

The first increment is 0.52 acres of freshwater marsh to be installed in the river channel. This increment is needed for mitigation and is inexpensive, with costs of $25,000 per acre. The second increment is 0.53 acres of marsh to be located in an excavated area in reach 12 adjacent to a new percolation pond. This increment is needed for mitigation but is more expensive, with a cost of $199,240 per acre. The third increment is 0.42 acres of the same type and location, but which is not needed for mitigation. The fourth increment is 0.15 acres, similar to the third increment, but more expensive due to additional excavation requirements caused by the site configuration. These increments add up to 1.62 acres, which is the total acreage of Bypass Channel Plan wetland mitigation that is not applied towards mitigating non-federal impact .

Table 3: Freshwater Marsh Mitigation Increments

ID	Description	Acreage	Cost	Cost/Acre
M-1	In-stream/needed/inexpensive	0.52	$13,000	$25,000
M-2	Off-stream/needed/expensive	0.53	$105,597	$199,240
M-3	Off-stream/not needed/expensive	0.42	$83,681	$199,240
M-4	Off-stream/not needed/more expensive	0.15	$45,802	$305,347

4.0 Evaluation of Increments

In this section, the cost-effectiveness of the above increments is compared within each habitat type and justified increments are selected for cost-sharing.

4.1 Riparian Forest/SRA Cover

Table 4 shows incremental costs associated with moving from less expensive to more expensive increments of riparian forest mitigation.

Increments R-1 through R-3 all have virtually the same cost per acre and presumably per habitat unit. Increment R-4 has a considerably higher per acre cost. However, this particular riparian forest mitigation unit would also provide 14.7 percent of the **total** SRA cover habitat units for the entire project (including existing SRA cover retained). Thus, this mitigation unit would be unusually effective in providing SRA cover, compared to other riparian forest mitigation units. Given the need to minimize impacts on the steelhead trout, and given the apparent shortfall of SRA habitat units after completion of all on-site riparian forest mitigation, this riparian forest mitigation unit should be regarded as essential. Therefore, all increments through R-4 are recommended as being cost-effective.

Table 4: Incremental Costs for Riparian Forest Mitigation

ID	Description	Acreage	Cost/ Acre	Incremental Cost/ Acre over First Increment	Incremental Cost/Acre over Previous Increm.
R-1	On site/needed/ inexpensive	14.60	$70,000	n/a	n/a
R-2	On site/extra/ inexpensive	4.03	$70,000	$0	$0
R-3	Off-site (reach13)/ extra/inexpensive	1.29	$71,000	$1,000	$1,000
R-4	On site (reach10b)/ extra/expensive	2.52	$288,254	$218,254	$217,254
R-5	Off-site (reach12)/ extra/expensive	2.00	$420,000	$350,000	$131,746

Increment R-5 has an even higher cost, and is not needed to mitigate project impacts. Furthermore, since the local sponsor's mitigation plan exhausts the potential for additional SRA cover habitat value within the study area, the installation of additional riparian forest mitigation in reach 12 or any other study reach would not create additional SRA cover. Therefore, this increment is not cost-effective and is not recommended.

4.2 Urban Forest

The proposed urban forest mitigation would probably produce habitat values somewhat less than those produced by riparian forest, but would also cost less per acre. These lands are not suitable for riparian forest mitigation. Since CESPK-RE has assessed all lands to be used for urban forest mitigation as channel lands essential for the project, since these lands have no other use if they are retained for the project, and since the per-acre cost of this mitigation increment is less than that of any of the riparian forest increments, this increment is determined to be cost-effective.

4.3 Freshwater Marsh

Table 5 shows the incremental costs associated with moving from less expensive to more expensive increments of freshwater marsh mitigation. Increment M-1 is very inexpensive and is cost-effective. Increment M-2 is much more expensive but is needed to achieve full mitigation. Its cost approximates the per acre cost of the most expensive riparian forest mitigation increment determined to be cost-effective; however, a direct comparison of efficiency of these mitigation increments is not possible. Increment M-3 has the same cost but is not needed to mitigate project impacts. Increment M-4 is the most expensive and is not needed to mitigate impacts.

Increments M-1 and M-2 are needed to achieve full mitigation of freshwater marsh losses. These increments are recommended for this reason, and because their costs are not unusually high compared to the riparian forest mitigation increments recommended above. Increment M-3 is recommended due to public and regulatory concerns over project impacts, and because it is no more expensive than increment M-2 and would be located on project lands. Increment M-4 is not recommended due to its higher cost and because it is not needed to mitigate impacts.

Table 5: Incremental Costs of Freshwater Marsh Mitigation

ID	Description	Acreage	Cost/Acre	Incremental Cost/Acre over First Increment	Incremental Cost/ Acre over Previous Increm.
M-1	Freshwater marsh/ needed/inexpensive	0.52	$25,000	n/a	n/a
M-2	Freshwater marsh/ needed/expensive	0.53	$199,240	$175,240	$175,240
M-3	Freshwater marsh/not needed/expensive	0.42	$199,240	$175,240	$0
M-4	Freshwater marsh/not needed/more expensive	0.15	$305,347	$280,347	$106,107

5.0 Conclusion

All riparian forest and urban forest mitigation proposed by the local sponsor is determined to be cost-effective and is recommended for cost-sharing. Total riparian forest/SRA mitigation recommended would cover 22.44 acres and would cost $2,122,090. Total urban forest mitigation recommended would cover 3.61 acres and would cost $216,600. Additional riparian forest mitigation would not be cost-effective. The riparian forest mitigation would compensate for nearly all SRA cover impacts. Additional measures such as removal of partial barriers to fish passage, installation of fish passage improvements, and upstream habitat improvements to be funded by the local sponsor, will provide an additional non-quantified amount of mitigation for SRA cover impacts.

All freshwater marsh mitigation needed to compensate for the impacts of the federal project is determined to be cost-effective and is recommended for cost-sharing. An additional small increment of freshwater marsh mitigation is proposed for cost-sharing as it can be accomplished at the same unit cost as the last increment of needed mitigation. The total acreage of freshwater marsh mitigation recommended is 1.47 acres with a cost of $202,278. The last increment of freshwater marsh mitigation proposed by the local sponsor is not recommended due to excessive costs and because it is not needed to mitigate for project impacts.

The total acreage of recommended mitigation is 27.52 acres with a cost of $2,540,968.

5.0 References

Army Corps of Engineers (1995). Evaluation of Environmental Investments Procedures Manual; Interim: Cost Effectiveness and Incremental Cost Analyses, IWR Report #95-R-1, May 1995.

U.S. Fish and Wildlife Service (1997). Revised Draft Coordination Act Report for the Guadalupe River Flood Control Project, Upper Reaches. Division of Ecological Services, Sacramento Field Office. April 1997.

REAL ESTATE

REAL ESTATE PLAN
UPPER GUADALUPE RIVER FLOOD PROTECTION STUDY
SANTA CLARA COUNTY, CALIFORNIA

INTRODUCTION

This report addresses the real estate requirements for the proposed flood control project for the upper Guadalupe River. This study was conducted under Section 4 of the Flood Control Act of 18 August 1941. A Reconnaissance report was prepared and completed in November 1989 for which a Real Estate Section was prepared. The Reconnaissance Report was approved in April 1990. The Feasibility Cost Sharing Agreement was signed in August 1990.

The Corps of Engineers' Sacramento District office completed a report in December 1991 entitled Guadalupe River General Design Memorandum. This project was authorized under Section 401(b) of the Water Resources Development Act of 1986 (Public Law 99-662), as amended by the Energy and Water Development Appropriations Act for Fiscal Year 1990 (Public Law 101-101). The Sacramento District is currently constructing portions of this project which are located in the vicinity of downtown San Jose and downstream of the area studied in this report. The alternatives studied in the Upper Guadalupe River Feasibility Study would pass flows through the downtown Guadalupe Flood Control Project.

There is no Federally owned land within the lands, easements, and rights-of-way required for this project.

The project's plan is to reduce damage to surrounding communities due to flooding from the upper Guadalupe River and its tributaries, Canoas Creek and Ross Creek. A secondary plan in this project is to include recreation features. It should be noted that there will be no additional lands required for the recreation features other than what is already required for the flood control project.

GENERAL DESCRIPTION OF THE PROJECT AREA

The project area is situated in an urban area of southwestern San Jose, in the extreme southern area of the Santa Clara Valley. Santa Clara Valley lies in the center of Santa Clara County in west central California, immediately south of San Francisco Bay. The county is bounded on the northeast by Alameda County, on the northwest by San Mateo County, and the southwest by Santa Cruz County. The area is highly urbanized. There are widespread residential subdivisions that are interspersed with commercial shopping centers, light industrial development, and some open spaces. The open spaces include three city-operated neighborhood parks, percolation ponds and a half-acre parcel used as a community garden, all owned by the Santa Clara Valley Water District (SCVWD). There are also water well fields owned by the San Jose Water Company adjacent to the

Guadalupe River. The study area has been divided into reaches, Reaches 7 through 13, in addition to Ross Creek and Canoas Creek. A description of the boundaries of each reach is in the main report.

GENERAL DESCRIPTION OF THE PROJECT AND TOTAL ACREAGE REQUIRED (Locally Preferred Plan and National Economic Development Plan)

Three alternative plans were formulated in addition to the no-action alternative for analysis in the Feasibility Report. They are the Willow Glen Plan, the Valley View Plan, and the Bypass Channel Plan. The National Economic Development (NED) Plan has been identified as the Valley View Plan because it maximizes the net benefits. However, the San Francisco District is recommending that the Bypass Channel Plan be constructed as the Recommended Plan. Following is the description of the NED and LPP Plans.

NED PLAN (Valley View Plan)

The Valley View Plan increases the minimum main stem capacity downstream of Canoas Creek to 12,000 cfs (providing sufficient channel capacity to convey the mean 50-year event. The improvements include channel widening and the replacement of the UPRR bridge in Reach 7, construction of low floodwalls, from one-to-three feet in height on the east and west banks of Reach 8, widening of the east bank near the Curtner Avenue Bridge, and replacement of the Hillsdale Bridge. All of the channel widening in this reach and upstream would be contained between the top of banks of the existing channel. Farther upstream, in Reaches 10 and 11, the channel would be widened on the appropriate banks given the specific flow constraints at that location.

Improvements to Ross Creek consist of widening the channel and construction of a 600-foot-long section of floodwall which would begin at Jarvis Avenue; and adding culverts at Almaden Expressway and Jarvis Avenue.

Improvements along Canoas Creek include adding culverts under streets and constructing low floodwalls on both banks.

This plan does not include any recreation features.

A total of 81.44 acres are required for this project. This will provide 25.38 acres for channel improvement easements for bypass channels and floodwalls, 13.88 acres for mitigation, 31.90 acres for flowage easements, and 10.28 acres for temporary work area easements. Some of the required lands are owned by the sponsor, the SCVWD, and some are owned by other public agencies. The SCVWD also has flood control easements over 2.42 acres of privately owned lands and over 1.55 acres of publicly owned lands. In addition, they have a maintenance easement over 1.08 acres of privately owned lands. There is, therefore, only 5.05 acres that are affected by easements out of the total 81.44 acres required for this project. The Appraisal Branch reviewed these easements and determined that these easements did not have a significant impact on the gross appraisal for this project at this phase. These easements will be considered in the site specific appraisals during acquisition of the lands for the project. The tables on the following pages detail the real estate requirements, ownerships, and values for this plan:

PROJECT FEATURE	ESTATE	ACREAGE/OWNERSHIPS	**EST. VALUE
Reach 7: (Southern Pacific RR river crossing to Union Pacific RR (UPRR)river crossing) Widen channel, construct floodwalls, replace 2 bridges	*CIE	Private -0.36 acs. SCVWD – 3.92 acs. SC County - 0.54 acs. State of CA - 1.69 acs. City of San Jose - 0.60 acs.	
Mitigation	Fee	Private -0.01 acs. SCVWD – 3.49 acs. SC County - 0.64 acs. State of CA - 1.20 acs. City of San Jose - 0.05 acs.	
Flowage	*FE	Private - 0.80 acs. SCVWD – 1.22 acs. SC County - 0.20 acs. State of CA - 2.21 acs. City of San Jose - 0.21 acs. TOTAL ACREAGE -17.14 acs. TOTAL OWNERSHIPS - 28 (23 Private and 5 public)	TOTAL $7,072,747
Reach 8: (UPRR) river crossing to Willow Glen Way) Construct floodwalls and replace bridge	CIE	Private - 2.66 acs. SCVWD – 1.89 acs.	
Mitigation:	Fee	Private - 0.65 acs.	
Flowage	*FEE	Private - 0.39 acs. TOTAL ACREAGE- 5.59 acs. TOTAL OWNERSHIPS - 21 (20 private and 1 public).	TOTAL $2,932,279

Reach 9: (Willow Glen Way to Curtner Avenue) No flood control structures.	CIE	0 acs.	
Mitigation	Fee	0 acs.	
Flowage	*FE	Private - 3.72acs. SCVWD - 1.55	
		TOTAL ACREAGE - 5.27 acs. TOTAL OWNERSHIPS - 65 (64 private and 1 public)	TOTAL $4,731,234
Reach 10A: (Curtner Avenue to Capitol Expressway) Channel widening	CIE	Private - 1.07 acs. SC County – 0.34 acs.	
Mitigation	Fee	Private - 0.22 acs. SCVWD - 0.16 SC County - 0.1	
Flowage	FE	Private - 0.82 acs. SCVWD - 0.37 acs City of San Jose – 0.41 acs.	
		TOTAL ACREAGE - 3.49 acs. TOTAL OWNERSHIPS – 19 (16 private and 3 public)	TOTAL $596,282
Reach 10B: (Curtner Avenue to Capitol Expressway) No flood control structures.	CIE	0 acs.	
Mitigation	Fee	Private - 0.03 acs. SCVWD - 2.35 acs. City of San Jose - 0.04 acs.	
Flowage	FE	Private - 0.39 SCVWD - 5.38 acs. City of San Jose - 0.35 acs	
		TOTAL ACREAGE - 8.54 acs. TOTAL OWNERSHIPS – 3 (1 PRIVATE AND 2 public)	TOTAL $1,013,119
Reach 10C: (Curtner Avenue to capitol Expressway) Widen channel and replace bridge.	CIE	Private - 1.55 acs. SCVWD - 1.05 acs. City of San Jose - 0.04 acs.	
Mitigation	Fee	Private - 0.39 acs. SCVWD - 1.00 acs. City of San Jose - 0.02 acs.	
Flowage	FE	Private - 0.80 acs. SCVWD - 1.68 acs. City of San Jose - 0.05 acs.	
		TOTAL ACREAGE - 6.58 acs. TOTAL OWNERSHIPS - 5 (3 private and 2 public).	TOTAL $4,577,316

Reach 11: (Capitol Expressway to Branham Lane) Widen channel.	CIE	Private -0.22 acs. SCVWD - 2.00 acs. SJ Water Co. - 0.67 acs.	
Mitigation	Fee	Private - 0.51 acs. SCVWD - 0.70 acs. SC County - 1.62 acs. SJ Water Co. - 0.28 acs.	
Flowage	FE	Private - 1.93 acs. SCVWD - 1.14 acs. SC County - 4.52 acs. SJ Water Co. – 0.41 acs. City of San Jose - 0.39 acs.	
		TOTAL ACREAGE - 14.39 acs. TOTAL OWNERSHIPS - 14 (10 private and 4 public)	TOTAL $987,034
Reach 12: (Branham Lane to Blossom Hill Road) No flood control features.	CIE	0 acs.	
Mitigation	Fee	SCVWD - 0.42 acs.	
Flowage	FE	SCVWD - 2.84 acs. SJ Water Co. - 0.12 acs.	
		TOTAL ACREAGES – 3.38 acs. TOTAL OWNERSHIPS - 2 (0 private AND 2 public)	TOTAL $13,454

Canoas Creek: Construct floodwalls and add culverts	CIE	SC County – 0.09 acs.	
Construct floodwalls	CIE	SCVWD – 0.36 acs.	
Construct Floodwalls and add culvert	CIE	SC County – 0.05 acs.	
Construct floodwall	CIE	SCVWD – 0.13 acs.	
Construction areas	*TWAE	SCVWD – 2.95 acs.	TOTAL $4,000.00

Ross Creek: Widen channel, new culverts	CIE	SCVWD – 6.15 acs.	
Construction Areas	TWAE	SCVWD – 7.33 acs.	TOTAL – 11,000.00

*CIE - Channel Improvement Easement
*FE - Flowage Easement
*TWAE - Temporary Work Area Easements
**Estimated Values in chart do not include contingencies, damages, and PL 91-646 Relocations. Those are included in the Baseline Cost Estimate.

LPP (Bypass Channel Plan)

The Bypass Channel Plan was formulated to provide approximately 100-year channel capacity to all the reaches. The bypass channel provides a means of conveying the excess flows above the existing channel capacity with the least amount of disruption to the existing river channel. This plan includes recreation features which have been incorporated into the design of the flood control features. Thus, no additional real estate is required for the recreation features. Each bypass channel is located immediately east of the existing channel. The largest bypass channel is located in Reaches 7 and 8 between Willow Street and Willow Glen Way. A low floodwall on the east side of the bypass is required for the channel upstream of Alma Street and downstream of the UPRR bridge. Reach 10 features consist of east bank widening where necessary and replacement of the Curtner and Hillsdale Avenue Bridges. Low floodwalls and a levee are also provided on the west bank from the Almaden Expressway southbound bridge to the Almaden expressway northbound bridge. Reach 11 features consist of channel widening and a bypass channel, where the preferred measure is a bypass channel. Low floodwalls are provided on the east bank in Reach 12. Improvements to Ross Creek would include increasing the bottom width of the channel to 35 feet and adding culverts at Almaden Expressway and Jarvis Avenue.

A total of 160.72 acres are required for this project providing 119.04 acres for channel improvement easements for bypass channels and floodwalls, 25.81acres for mitigation, 5.59 acres for recreation, and 10.28 acres for temporary work area easements. Some of the required lands are owned by the sponsor, the SCVWD, and some are owned by other public agencies. The same easements as discussed in the NED Plan description apply in this plan.. The tables on the following pages detail the real estate requirements, ownerships, and values for this plan:

PROJECT FEATURE	ESTATE	ACREAGE/OWNERSHIPS	**EST. VALUE
Reach 7: (Southern Pacific RR river crossing to Union Pacific RR (UPRR)river crossing) Construct bypass channels and floodwalls	*CIE	Private -1.57 acs. SCVWD – 0.70 acs. SC County - 0.23 acs. State of CA - 2.79 acs.	
Mitigation	Fee	Private -1.20 acs. SCVWD – 0.43 acs. SC County - 0.75 acs. State of CA - 2.60 acs.	
Recreation	Fee	Private - 0.09 acs. SCVWD – 0.09 acs. SC County - 0.06 acs. State of CA - 0.51 acs. Unknown .(no parcel no.) - 0.03 acs. TOTAL ACREAGE -11.05 acs. TOTAL OWNERSHIPS - 30 (23 Private and 7 public)	TOTAL $3,913,520
Reach 8: (UPRR) river crossing to Willow Glen Way) Channel widening and construct bypass channel	CIE	Private - 4.23 acs. SCVWD – 3.62 acs.	
Mitigation:	Fee	Private - 0.10 acs. SCVWD – 0.22 acs. City of SJ – 0.06	
Recreation	Fee	Private - 0.31 acs. SCVWD – 0.38 acs TOTAL ACREAGE- .8.92 acs. TOTAL OWNERSHIPS - 40 (31 private and 9 public).	TOTAL $4,812,370

Reach 9: (Willow Glen Way to Curtner Avenue) Channel widening and construct bypass channel	CIE	Private - 7.62 acs. SCVWD - 3.06 acs. SJ Water Co. - 1.22 acs.	
Mitigation	Fee	Private -1.03 acs. SCVWD - 0.72 acs Santa Clara County - 0.22 acs. San Jose Water Co. - 0.15	
Recreation	Fee	0 acs..	
		TOTAL ACREAGE - 14.02 acs. TOTAL OWNERSHIPS - 81 (74 private and 7 public)	TOTAL $7,955,731
Reach 10A: (Curtner Avenue to Capitol Expressway) Channel widening	CIE	Private - 2.77 acs. SCVWD - 0.47 acs. SC County – 0.49 acs.	
Mitigation	Fee	Private - 0.22 acs.	
Recreation	Fee	Private - 0.02 acs. SCVWD - 0.06 acs Unknown.-(no APN no) . – 0.02 acs.	
		TOTAL ACREAGE - 4.05 acs. TOTAL OWNERSHIPS – 23 (19 private and 4 public)	TOTAL $1,170,875
Reach 10B: (Curtner Avenue to Capitol Expressway) Construct levees and floodwalls	Flood Protection Levee Easement	Private - 1.94 acs. SCVWD - 10.37 acs.	
Mitigation	Fee	Private - 0.46 acs. SCVWD - 2.24 acs.	
Recreation	Fee	SCVWD - 0.93 acs.	
		TOTAL ACREAGE - 15.94 acs. TOTAL OWNERSHIPS – 10 (1 PRIVATE AND 9 public)	TOTAL $2,180,092
Reach 10C: (Curtner Avenue to capitol Expressway) Widen channel	CIE	Private - 4.19 acs. SCVWD - 5.99 acs.	
Mitigation	Fee	Private - 1.67 acs. SCVWD - 1.12 acs.	
Recreation	Fee	Private - 0.54 acs. SCVWD - 0.10 acs. Unknown (.-No. APN no.) - 0.04 acs.	
		TOTAL ACREAGE - 13.65 acs. TOTAL OWNERSHIPS - 2 (11 private and 10 public).	TOTAL $6,898,880

Reach 11: (Capitol Expressway to Branham Lane) Widen channel, construct bypass channel	CIE	Private -5.54 acs. SCVWD - 5.62 acs. SC County -6.14 acs. SJ Water Co. - 1.86 acs.	
Mitigation	Fee	Private - 0.44 acs. SCVWD - 1.44 acs. SC County - 2.15 acs. SJ Water Co. - 0.20 acs.	
Recreation	Fee	Private - 0.36 acs. SCVWD - 0.73 acs. SJ Water Co. – 0.03 acs.	
		TOTAL ACREAGE - 24.51 acs. TOTAL OWNERSHIPS - 38 (19 private and 19 public)	TOTAL $2,184,197
Reach 12: (Branham Lane to Blossom Hill Road) Channel widening	CIE	Private - 5.94 acs. SCVWD - 45.99 acs. SJ Water Co. - 0.62 acs.	
Mitigation	Fee	Private - 3.00 acs. SCVWD - 3.97 acs. San Jose Water Co. - .13 acs.	
Recreation	Fee	Private - 0.02 acs. SCVWD - 1.11 acs. SJ Water Co. - 0.16 acs.	
		TOTAL ACREAGES – 60.94 acs. TOTAL OWNERSHIPS - 31 (8 private AND 23 public)	TOTAL $7,271,455
Reach 13: Mitigation	Fee	SCVWD - 1.29 acs. TOTAL ACREAGE - 1.29 TOTAL OWNERSHIPS - 1	TOTAL $561.920

Canoas Creek: Construct floodwalls and add culverts	CIE	SC County – 0.09 acs.	
Construct floodwalls	CIE	SCVWD – 0.36 acs.	
Construct Floodwalls and add culvert	CIE	SC County – 0.05 acs.	
Construct floodwall	CIE	SCVWD – 0.13 acs.	
Construction areas	*TWAE	SCVWD – 2.95 acs.	TOTAL $4,000.00

Ross Creek: Widen channel, new culverts	CIE	SCVWD – 6.15 acs.	
Construction Areas	TWAE	SCVWD – 7.33 acs.	TOTAL – 11,000.00

*CIE - Channel Improvement Easement

*TWAE - Temporary Work Area Easement

**Estimated Values in chart do not include contingencies, damages, and PL 91-646 Relocations. Those are included in the Baseline Cost Estimate.

ESTATES

The non-Federal sponsor will acquire the minimum interest in real estate to support the construction, operation, and maintenance of the project. The estates for both the NED and LPP plans are as follows:

(NED) - The minimum estates for the NED Plan are channel improvement easements, temporary work area easements for the construction areas, staging areas and temporary access roads, fee for mitigation lands, and flowage easements. There is also a requirement for mitigation lands to be provided in conjunction with the flood control aspect of the project. Lands that would satisfy the mitigation requirement are located within the project boundaries required for flood control. All lands that provide for multiple purposes, such as channel improvement work and mitigation, shall require the fee estate. Lands for the channel improvement work were valued at fee value. Those lands that are required only for flood control will require the channel improvement easement estate.

(LPP) - The minimum estates for the LPP are channel improvement easements, temporary work area easements for the construction areas, staging areas and temporary access roads, and fee for mitigation lands. There is a requirement for mitigation lands to be provided in conjunction with the flood control aspect of the project. Lands that would satisfy the mitigation requirement are located within the project boundaries required for flood control. In addition, there will be a recreation trail on roads that are also within the project boundaries. All lands that provide for multiple purposes, such as channel improvement work, mitigation, and recreation shall require the fee estate. Lands for the channel improvement work were valued at fee value. Those lands that are required only for flood control will require the channel improvement easement estate.

BASELINE COST ESTIMATE (NED AND LPP)

The Appraisal Branch of the Sacramento District Real Estate Division prepared the gross appraisals for the Feasibility Report at October 1, 1997 price levels. The gross appraisal for the LPP was approved by HQ USACE CERE-A on 22 March 1997. A gross appraisal for the NED was prepared and approved on 23 February 1998. The non-Federal sponsor prepared their estimate of acquisition costs and schedules based on their knowledge of project requirements and anticipated staffing and resource levels. All lands have been estimated at fair market value. The differences between State and Federal appraisal rules have been considered. There is no difference between the application of State and Federal rules in the valuation of the lands to be acquired.

NED Baseline Cost Estimate

Project	Non-Federal	Federal	*Land (LERRDS)	Project Cost
Upper Guadalupe (NED)	$3,028,084	$378,133	$27,259,306	$30,700,000
Project Total	$3,028,084	$378,133	$27,259,306	$30,700,000

***Includes Lands, Damages, and Contingencies but does not include damages and contingencies for publicly owned lands. Public Law 91-646 Relocations, estimated at $898,000, are included in the LERRDs costs.**

LPP Baseline Cost Estimate

Project	Non-Federal	Federal	*Land (LERRDS)	Project Cost
Upper Guadalupe (LPP)	$3,575,300	$457,300	$46,000,000	$50,032,600
Project Total	$3575,300	$457,300	$46,000,000	$50,032,600

***Includes Lands, Damages, and Contingencies but does not include damages and contingencies for publicly owned lands. Public Law 91-646 Relocations, estimated at $898,000, are included in the LERRDS costs.**

THE NON-FEDERAL SPONSOR'S ABILITY TO ACQUIRE

The non-Federal sponsor has a real estate staff and experience in acquiring real estate for County, State, and Federal projects. They have not previously received credit for lands described in this report and required for this project. The sponsor owns about 40 or 50 residential properties within the project boundaries that they have been purchasing sine 1984. Standard crediting procedures are expected to be applied to these acquisitions. There are no known special crediting issues for any of the land for this project. The sponsor has been advised of the risks of acquiring property prior to the execution of the PCA.

The sponsor has condemnation authority. Chapter 1405 of the Statutes of 1951: Santa Clara Valley Water District Act (As Amended) declares that "...The district may exercise the right of eminent domain, either within or without said district, to take any property necessary to carry out any of the objects or purposes of this act. ...". An Assessment of Non-Federal Sponsor's Real Estate Acquisition Capability is being forwarded with the Draft Feasibility Report which discusses the sponsor's ability to acquire in more detail.

PUBLIC LAW (91-646) UNIFORM RELOCATIONS ASSISTANCE AND REAL PROPERTY ACQUISITION POLICIES ACT OF 1970 as amended by PUBLIC LAW 100-17

There are approximately 9 owner-occupant residences, 4 renter occupants, and 16 businesses that will require relocation. This applies to both the NED and the LPP. The sponsor has prepared a Relocation Plan. The sponsor is aware of and has experience with PL 91-646 requirements. The total cost for these relocations has been estimated at $898,000 and is included in the baseline cost estimate. This cost provides for $50,000 per owner-occupied residence, $12,000 per tenant, and $25,000 per business. These costs include payment of Last Resort Housing Benefits. Justification for Last Resort Housing

Benefits will be provided as required by 24 C.F.R., 24.404 (1992) on a case by case basis.

FACILITY AND UTILITY RELOCATIONS

There are numerous utility and facility relocations in this project for both the NED Plan and LPP. They consist of bridge removals and replacements, relocation/modifications of sanitary sewer, water lines, stormwater outfalls, relocation of booster pumps and two wells, two temporary railroad relocations, and two road replacements for culvert construction. These "relocations" have been researched and are considered at this time to be compensable since the facilities and utilities are there by an interest in real estate and there is an ongoing requirement for them. Following is information on the relocations:

Bridges - There are five bridges that will be affected by the project. They are the Willow Street Bridge in Reach 7A, Alma Avenue Bridge in Reach 7B, Willow Glen Bridge in Reach 8, Curtner Avenue Bridge in Reach 10A, and Hillsdale Avenue Bridge in Reaches 10 C&D. (**NOTE**: Curtner Bridge is only to be relocated in the LPP and not required to be relocated for the NED Plan.) The work to be done on the bridges mainly consists of removing and replacing them at a higher elevation to accommodate the project design. The bridge replacement would require the same road width with no parking. Any betterment would be cost shared between the sponsor and the City of San Jose.

Malone Street Bridge – The Malone Street Bridge discussed in the main report, located in Reach 9, is not a relocation. The work in relation to the Malone Street Bridge involves a retaining wall which will serve to protect an access road that will be constructed as a project feature. Based on this, the Malone Street Bridge work is considered to be a construction cost. (**NOTE**: The Malone Street Bridge work is not required in the NED Plan.)

Hillsdale Avenue Bridge – The Hillsdale Avenue Bridge is considered to be a "relocation" as defined in WRDA, although there is a change in the location of the bridge. An Attorney's Opinion of Compensability (AOC) has been prepared that confirms that there is an ongoing necessity of this bridge notwithstanding the change in the location. The new location of the bridge is approximately 2,200 feet downstream of its current location. The bridge is being relocated for efficiency of traffic load management. The bridge will serve the same people it serves in its present location. The AOC is presently being forwarded to HQUSACE.

Utility Relocations (Water and Sewer Lines, Stormwater Outfalls, Booster Pumps, and Wells) - These utilities and facilities will be re-routed and/or raised or lowered to accommodate the new bypass channel and design. Any betterment would be cost shared between the sponsor and the City of San Jose and/or San Jose Water Company.

Road Replacements - These roads are currently used, will continue to be used, and are owned by the Santa Clara County Transportation District. These modifications consist of in-kind road replacements. There will be detours or rerouting. The costs associated with the traffic delays have been computed and included in the cost analysis as represented in the Economics Appendix to the main report. There are no real estate impacts such as requirements for additional lands.

"ANY CONCLUSION OR CATEGORIZATION CONTAINED IN THIS REPORT THAT AN ITEM IS A UTILITY OR FACILITY RELOCATION TO BE PERFORMED BY THE NON-FEDERAL SPONSOR AS PART OF ITS LERRD RESPONSIBILITIES IS PRELIMINARY ONLY. THE GOVERNMENT WILL MAKE A FINAL DETERMINATION OF THE RELOCATIONS NECESSARY FOR THE CONSTRUCTION, OPERATION, OR MAINTENANCE OF THE PROJECT AFTER FURTHER ANALYSIS AND COMPLETION OF FINAL ATTORNEY'S OPINIONS OF COMPENSABILTY FOR EACH OF THE IMPACTED UTILITIES AND FACILITIES."

RECREATION (LPP ONLY)

The recreation feature proposed for this project is a multi-use recreation trail linking the study area with the existing trails along the Guadalupe River in downtown San Jose and upstream of Blossom Hill Road. There will not be a separate PCA for the recreation aspects of this project, and the SCVWD will be the sponsor for the recreation in this project as well as the flood control project. The design of the recreation trail has been coordinated with the City of San Jose, and brief stretches of the trail will be located off-site but will be the sole responsibility of the City of San Jose. The land required for recreation will be constructed on project lands that are required for flood control purposes. The recreation feature in this project is a recreation trail that will be located on the surface of gravel-based maintenance access roads which are required for the proposed project. The trail will follow the maintenance road and portions of the bypass channels and levees within the project area. In order to provide a continuous pathway, portions of the trail will leave the project lands and will be provided off-site on city streets by the City of San Jose. These off-site areas will be provided by the City of San Jose in coordination with the non-Federal sponsor and at no cost to the project. The off-site areas will provide continuity to the recreation features in the project. Additional recreation features, such as picnic areas and bathrooms, will be included in the overall recreation plan. These additional features will be located in Reaches 7, 10b, 11, and 12 and require only minimal areas. They will be located off the maintenance access roads and on lands that have appraised for a channel improvement easement and for which the gross appraisal determined fee value. As stated in the Estate Section of this report, lands that provide for multiple purposes such as in this case where recreation and a maintenance road are required, the fee estate will be required

The recreation areas are located in the following reaches:

REACH	ACREAGE	OWNER	*VALUE
7	0.09 acres	Private	“0”
	0.09 “	SCVWD	“0”
	0.06 “	SC County	
	0.51 “	State of CA	
	0.03 “	Unknown	
8	0.31 “	Private	“0”
	0.38 “	SCVWD	“0”
10A	0.02 “	Private	“0”
	0.06 “	SCVWD	“0”
	0.02 “	No APN	“0”
10B	0.93 “	SCVWD	“0”
10C	0.54 “	Private	“0”
	0.10 “	SCVWD	“0”
	0.04 “	No APN No.	“0”
11	0.36 “	Private	“0”
	0.73 “	SCVWD	“0”
	0.03 “	SJ Water Co.	“0”
12	0.02 “	Private	“0”
	1.11 “	SCVWD	“0”
	0.16 “	SJ Water Co.	“0”

*There is no land value attributed to the recreation features in his project. This is because the recreation features are included in the proposed flood control project for flood control purposes. The channel improvement easements for these flood control areas were valued at full fee value, and there is no residual value for recreation.

MITIGATION

LPP - There is a total of 27.52 acres of mitigation lands of which 22.174 acres for riparian forest, 3.604 acres for urban forest, and 2.578 acres for wetland. These lands are within the channel, with the exception of 2.234 acres of urban forest that are outside the channel but still within the 160.72 acres described in this report.

NED - There is a total of 12.1 acres of riparian forest mitigation lands for the NED Plan. These lands are mostly within the channel.

ATTITUDE OF LANDOWNERS

The landowners in the project area are generally in favor of the project. There are no anticipated major or specific issues regarding acquisition of property with the landowners outside of normal negotiations for property acquisition.

MINERALS

There are no valuable minerals impacted by this project and thus no subsurface minerals or timber activity, and therefore no enhancement for mineral deposits is included in the baseline cost estimate.

HAZARDOUS, TOXIC, AND RADIOLOGICAL WASTE (HTRW)

A project area review and preliminary assessment (PSA) of the study areas was conducted in 1992. There were eight sites noted as potential HTRW sites, which contain mercury, nickel and silver, petroleum hydrocarbons, and pesticides. The potential causes are businesses such as automotive repair, etc. The sponsor will be responsible for cleanup of lands, and such cleanup will be done prior to construction. This issue is described in more detail in the main report. The appraiser was not knowledgeable of the nature and extent of the contamination nor the cost of the removal and clean up. Therefore, the sites valued were treated as if no contamination or toxic concerns exist with the assumption that any negotiations for acquisition will consider the cost of treating the toxic contaminants.

ACQUISITION SCHEDULE

The Acquisition Schedule is included as Exhibit A.

PROJECT MAPS

Project maps are included as Exhibit B.

Real Estate Milestones Project Name				
	COE Start	COE Finish	NFS Start	NFS Finish
Receipt of Final Drawings from Engineering/Project Manager		3/00		
Execution of PCA		3/00		3/00
Formal Transmittal of Final ROW Drawings and Instruction to Acquire LERRD		4/00		
Conduct Landowner Meetings			4/00	4/00
Prepare/Review Mapping and Legal Descriptions			4/00	5/00
Obtain/Review Title Evidence			4/00	5/00
Obtain/Review Tract Appraisals			5/00	7/00
Conduct Negotiations			7/00	11/01
Perform Closings			8/00	11/01
Prepare/Review Condemnations			06/01	11/01
Perform Condemnations			07/01	11/01
Obtain Possession			09/01	11/01
Complete/Review PL 91-646 Benefit Assistance			09/01	11/01
Conduct/Review Facility and Utility Relocations			09/01	11/01
Certify All Necessary LERRD for Construction	11/01	12/01		
Prepare and Submit Credit Requests			12/01	3/02
Review and Approve or Deny Credit Requests	12/01	5/02		
Establish Value for Creditable LERRD in F&A Cost Accounting System	05/02	06/02		

COE Engineers
NFS - Non-Federal Sponsor

EXHIBIT A

ASSESSMENT OF NON-FEDERAL SPONSOR'S REAL ESTATE ACQUISITION CAPABILITIES

UPPER GUADALUPE RIVER FLOOD PROTECTION STUDY

I. Legal Authority:

a. Does the sponsor have legal authority to acquire and hold title to real property for project purposes? **YES**

b. Does the sponsor have the power of eminent domain for this project? **YES**

c. Does the sponsor have "quick-take" authority for this project? **NO**

d. Are any of the lands/interests in land required for the project located outside the sponsor's political boundary? **NO**

e. Are any of the lands/interests in land required for the project owned by an entity whose property the sponsor cannot condemn? **NO**

II. Human Resource Requirements

a. Will the sponsor's in-house staff require training to become familiar with the real estate requirements of Federal projects including PL 91-646, as amended? **NO**

b. If the answer to II.a. is "yes", has a reasonable plan been developed to provide such training? **N/A**

c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? **YES**

d. Is the sponsor's projected in-house staffing level sufficient considering its other workload, if any, and the project schedule? **YES**

e. Can the sponsor obtain contractor support, if required, in a timely fashion? **YES**

f. Will the sponsor likely request USACE assistance in acquiring real estate? **NO**

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? **YES**

b. Has the sponsor approved the project/real estate schedule/milestones? **YES**

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? **YES**

b. With regard tot his project, the sponsor is anticipated to be **HIGHLY CAPABLE.**

V. Coordination:

a. Has this assessment been coordinated with the sponsor? **YES**

b. Does the sponsor concur with this assessment? **YES**

Prepared by:

Susan Miller

SUSAN MILLER
Real Estate Manager

Reviewed and approved by:

MARVIN D. FISHER
Chief, Real Estate Division

DATE 9-4-97

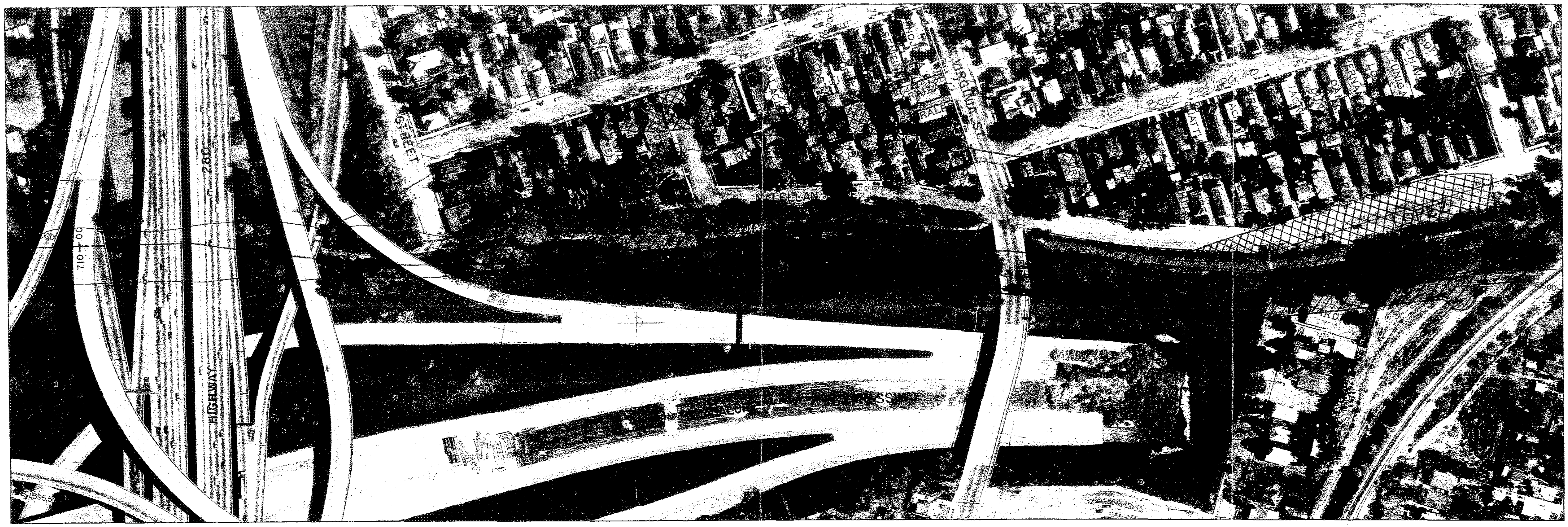
EXHIBIT B
REAL ESTATE MAPS

(Sheets 9-20)

Note that there are no Sheets 1-8

EXHIBIT B

BYPASS CHANNEL PLAN PROJECT MAPS



▒	SCVWD PROPERTY
▒	OTHER PROPERTIES TO BE ACQUIRED
	SCVWD EASEMENT
———	PROPOSED R/W (PROJECT LIMIT LINE)
— — —	EXISTING TOP-OF-BANK
▨	MAINTENANCE EASEMENT NEEDED ONLY
▩	PROPOSED MITIGATION AREA

Return to SCVWD

Up to date as of Nov. [illegible] TJ1/PRB

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:

Right-of-Ways at Tops of Bank.

PHOTOGRAPHY BY: AIR FLIGHT SERVICE DATE: 2-27-85 RECTIFICATION BY: AERYTEK, INC. 84031

DATE

DESIGN

DRAWN

CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

PROJECT ENGINEER R.C.E. NO.

DIVISION ENGINEER R.C.E. NO.

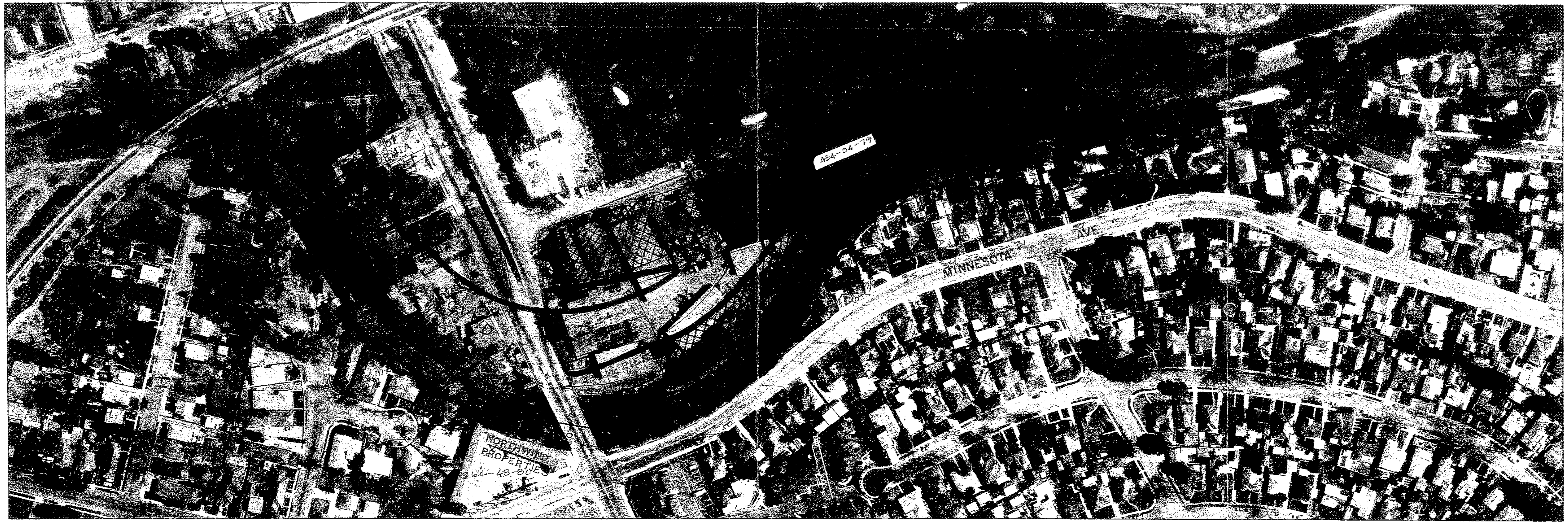
ENGINEERING APPROVAL

HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE 1" = 100'

SHEET 9



▨ SCVWD PROPERTY

▨ OTHER PROPERTIES TO BE ACQUIRED

SCVWD EASEMENT

PROPOSED R/W

EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT NEEDED ONLY

▩ PROPOSED MITIGATION AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:

△ 20 N 299,548.58 E 1,596,084.90 EL 108.63
○ 217 EL 108.45

PHOTOGRAPHY BY: AIR FLIGHT SERVICE DATE: 2-27-85 RECTIFICATION BY: AERO-TEK, INC. 84031

DATE	
DESIGN	
DRAWN	
CHECKED	

Santa Clara Valley Water District

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DIVISION ENGINEER R.C.E. NO.

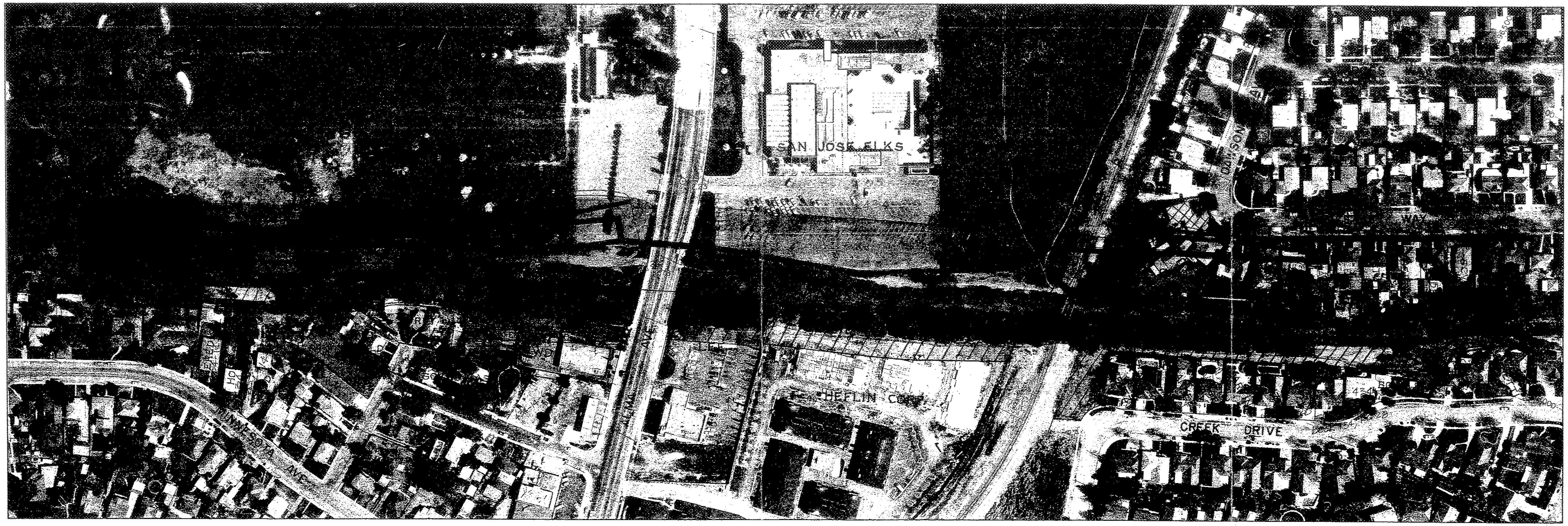
ENGINEERING APPROVAL

HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE 1" = 100'

SHEET 10



SCVWD PROPERTY

OTHER PROPERTIES
TO BE ACQUIRED

SCVWD EASEMENT

PROPOSED R/W

EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT
NEEDED ONLY

▩ PROPOSED MITIGATION
AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:

△22 N 297,755.46 E 1,597,430.45 EL 112.78
⊙219 EL 112.23

PHOTOGRAPHY BY: AIR FLIGHT SERVICE DATE: 2-27-85 RECTIFICATION BY: AELYTEC, INC. 84031

DATE
DESIGN
DRAWN
CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION
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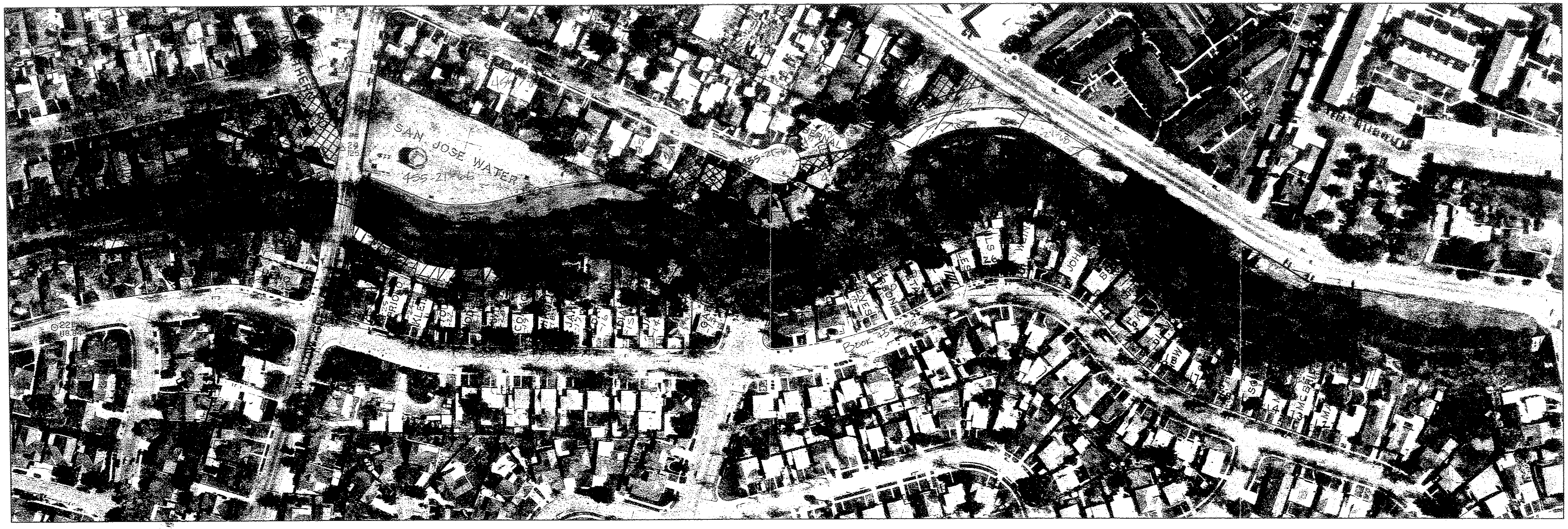
DIVISION ENGINEER R.C.E. NO.

ENGINEERING APPROVAL
HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE
1" = 100'

SHEET
11



▒ SCVWD PROPERTY

▒ OTHER PROPERTIES TO BE ACQUIRED

SCVWD EASEMENT

——— PROPOSED R/W

– – – EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT NEEDED ONLY

▩ PROPOSED MITIGATION AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:

△24 N 295,725.57 E 1,598,423.86 EL 125.07
⊙222 EL 125.65

PHOTOGRAPHY BY: AIR FLIGHT SERVICE DATE: 2-27-85 RECTIFICATION BY: AERO-TEC, INC. 84031

DATE	
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Santa Clara Valley Water District

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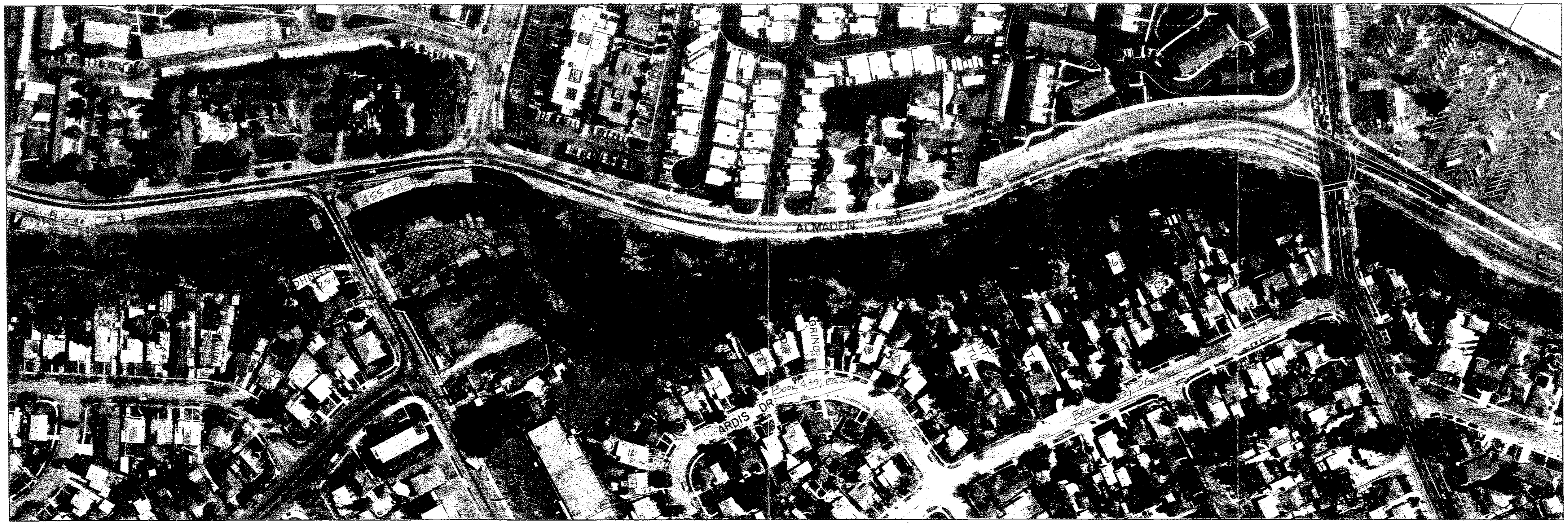
ENGINEERING APPROVAL

HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE 1" = 100'

SHEET 12



▨ SCVWD PROPERTY

▨ OTHER PROPERTIES TO BE ACQUIRED

▨ SCVWD EASEMENT

——— PROPOSED R/W

– – – EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT NEEDED ONLY

▨ PROPOSED MITIGATION AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:

△25 N 293,210.34 E 1,598,400.49 EL 133.55

△28 N 291,419.62 E 1,598,562.98 EL 142.53

PHOTOGRAPHY BY: AIR FLIGHT SERVICE

DATE: 2-27-85

RECTIFICATION BY: AELYTEK, INC. 84031

DATE	
DESIGN	
DRAWN	
CHECKED	

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

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DIVISION ENGINEER R.C.E. NO.

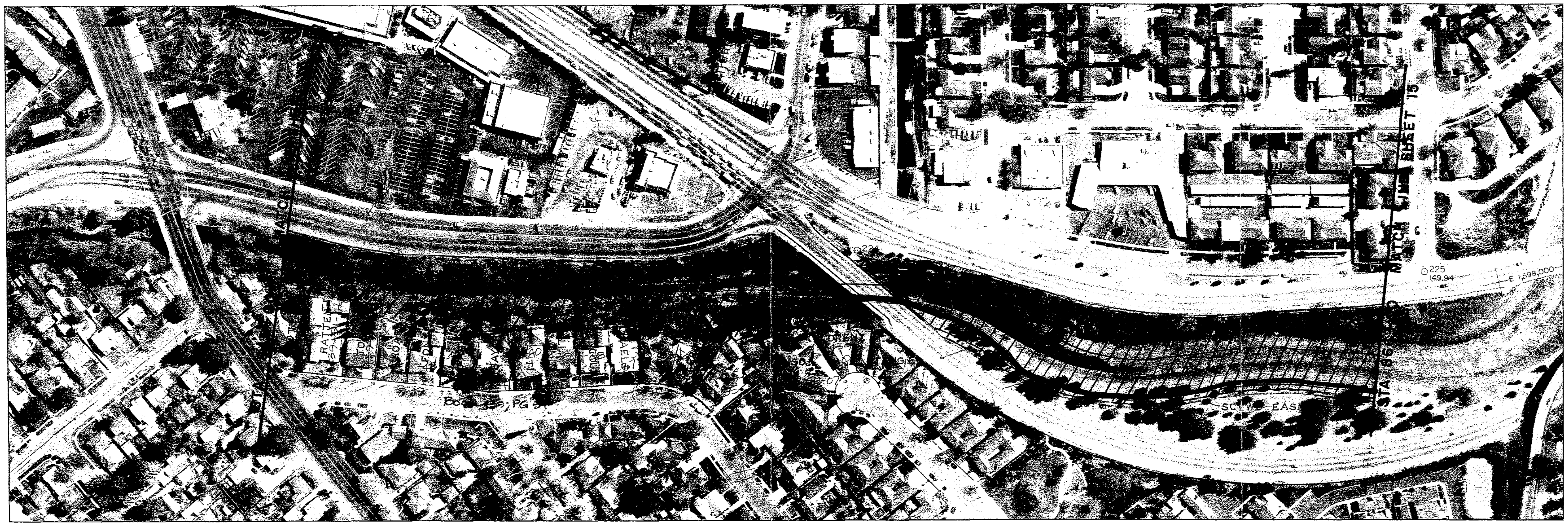
ENGINEERING APPROVAL

HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE 1" = 100'

SHEET 13



SCVWD PROPERTY

OTHER PROPERTIES TO BE ACQUIRED

SCVWD EASEMENT

——— PROPOSED R/W

– – – EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT NEEDED ONLY

▩ PROPOSED MITIGATION AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:
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⊙223 EL 143.04
⊙224 EL 147.77
⊙225 EL 149.94

PHOTOGRAPHY BY: AIR FLIGHT SERVICE DATE: 2-27-85 RECTIFICATION BY: AEROTEC, INC. 84031

DATE	
DESIGN	
DRAWN	
CHECKED	

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

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DIVISION ENGINEER R.C.E. NO.

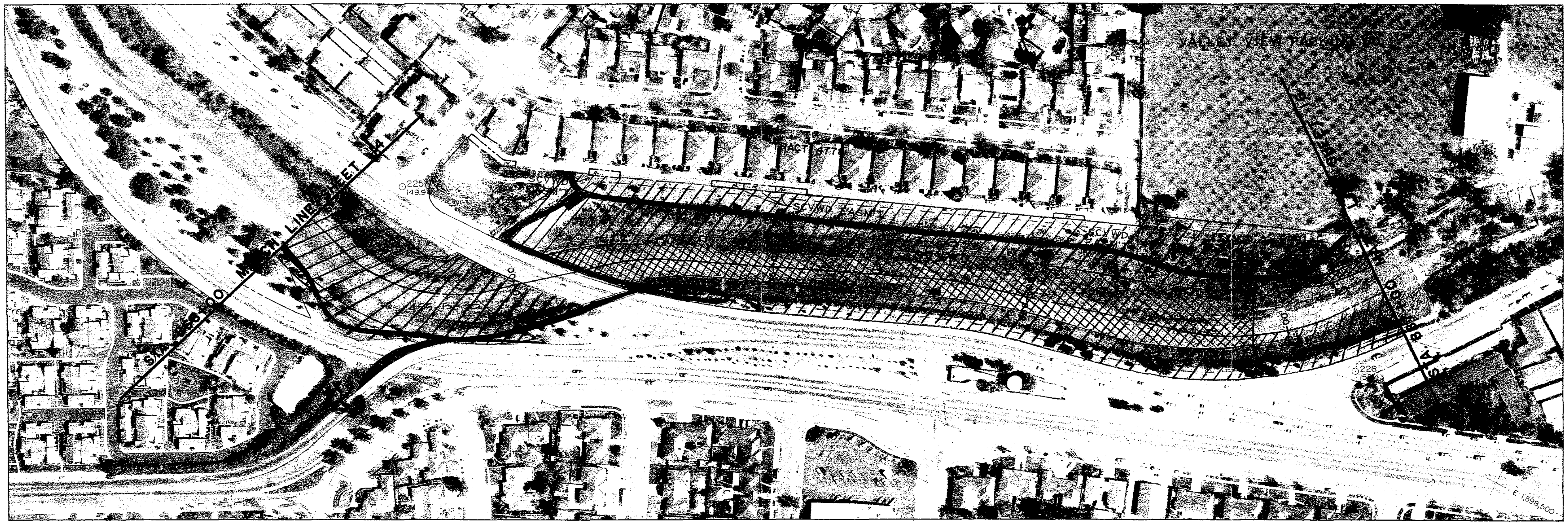
ENGINEERING APPROVAL

HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE 1" = 100'

SHEET 14



░	SCVWD PROPERTY
░	OTHER PROPERTIES TO BE ACQUIRED
	SCVWD EASEMENT
——	PROPOSED R/W
– – –	EXISTING TOP-OF-BANK
▨	MAINTENANCE
▩	
▬	RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:

○ 225 EL 149.94
○ 226 EL 155.43

PHOTOGRAPHY BY: AIR FLIGHT SERVICE
DATE: 2-27-85
RECTIFICATION BY: AEROTEC, INC. 84031

DATE
DESIGN
DRAWN
CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION
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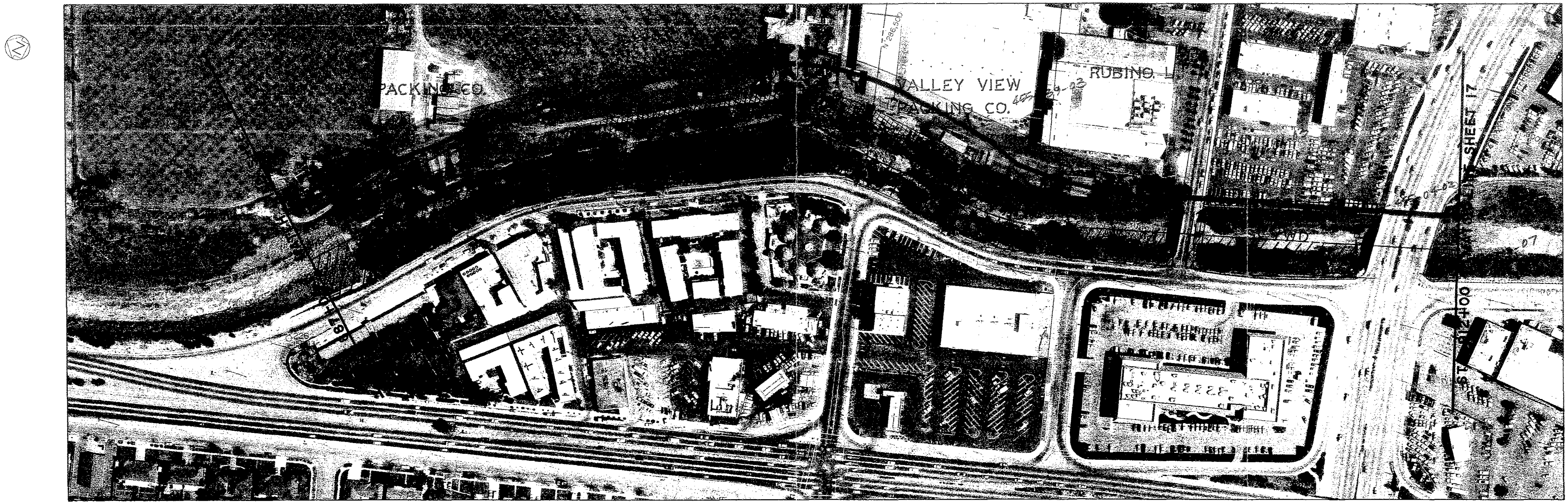
DIVISION ENGINEER R.C.E. NO.

ENGINEERING APPROVAL
HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE
1" = 100'

SHEET
15



SCVWD PROPERTY

OTHER PROPERTIES
TO BE ACQUIRED

SCVWD EASEMENT

——— PROPOSED R/W

— — — EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT
NEEDED ONLY

▩ PROPOSED MITIGATION
AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:

△29 N 285,775.43 E 1,599,432.11 EL 159.93
⊙226 EL 155.43
⊙227 EL 157.58

PHOTOGRAPHY BY: AIR FLIGHT SERVICE

DATE: 2-27-85

RECTIFICATION BY: AEROTEK, INC. 84031

DATE	
DESIGN	
DRAWN	
CHECKED	

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

PROJECT ENGINEER R.C.E. NO.

DIVISION ENGINEER R.C.E. NO.

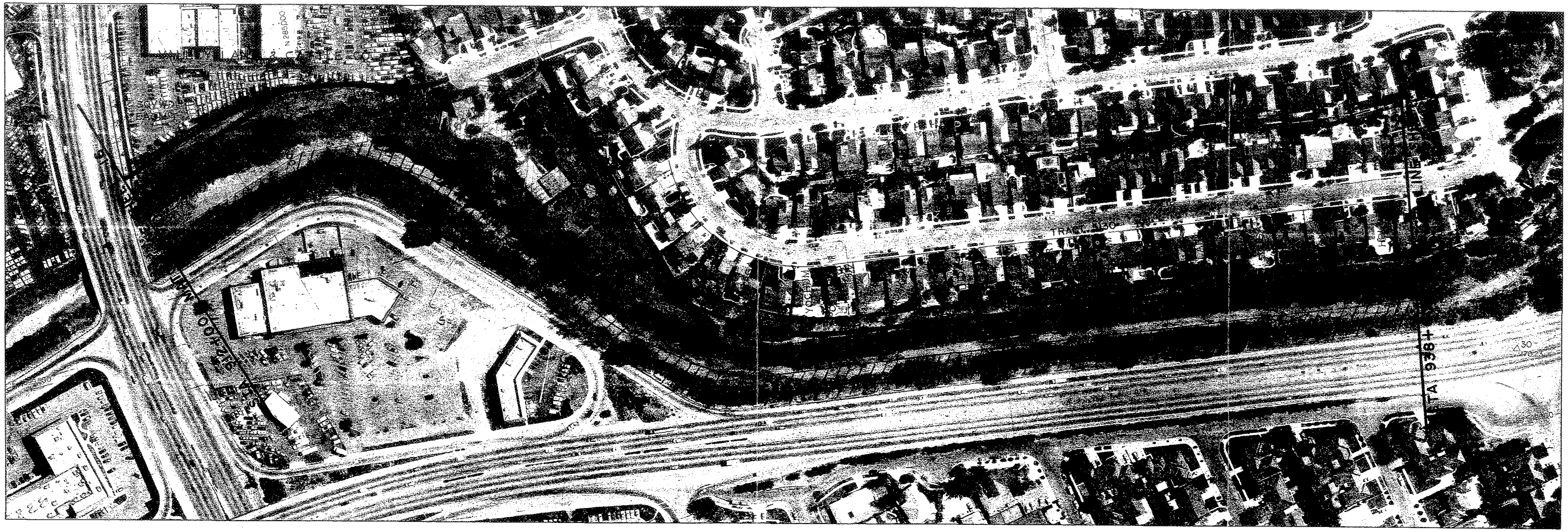
ENGINEERING APPROVAL

HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE
1" = 100'

SHEET
16



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▒ OTHER PROPERTIES TO BE ACQUIRED

▒ SCVWD EASEMENT

— PROPOSED R/W

— EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT NEEDED ONLY

▩ PROPOSED MITIGATION AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:

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⊙ 227 EL 157.58

PHOTOGRAPHY BY: AIR FLIGHT SERVICE

DATE: 2-27-85

RECTIFICATION BY: AERTEC, INC. 84031

DATE	
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CHECKED	

Santa Clara Valley Water District

ENGINEERING CERTIFICATION

PROJECT ENGINEER R.C.E. NO.

DIVISION ENGINEER R.C.E. NO.

ENGINEERING APPROVAL

HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE 1" = 100'

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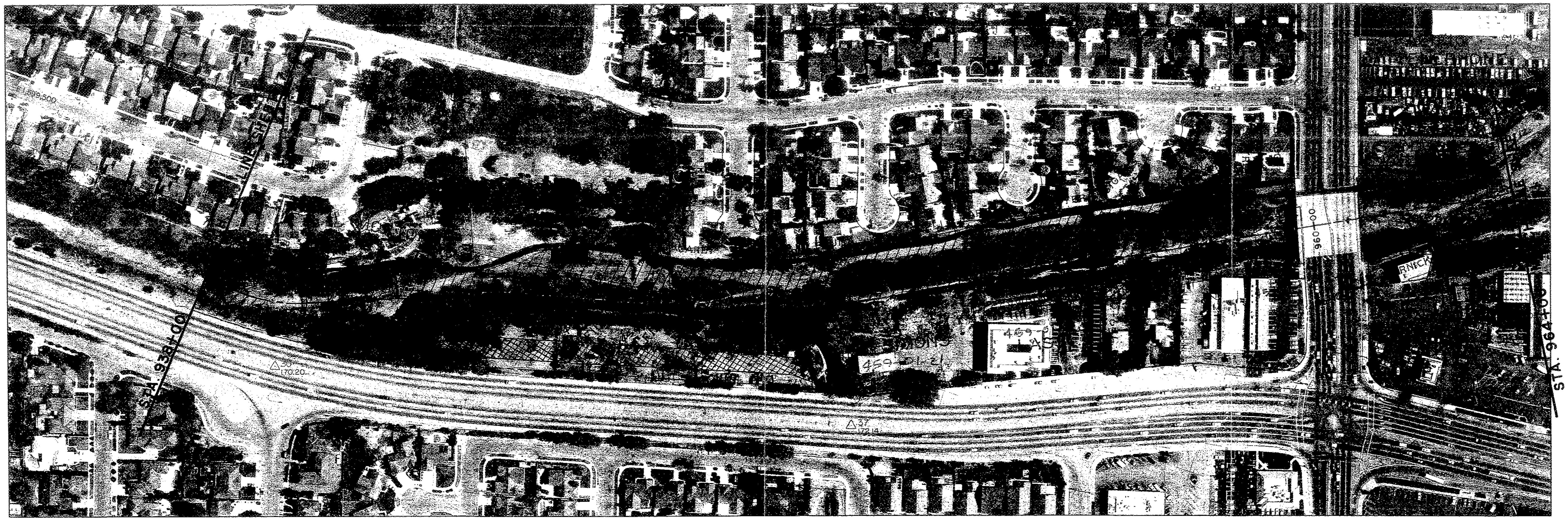

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▩ PROPOSED MITIGATION AREA

▬ RECREATION TRAIL

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△ 37 N 281,652.20 E 1,599,378.15 EL 172.14
○ 228 EL 168.42
○ 229 EL 171.91

PHOTOGRAPHY BY: AIR FLIGHT SERVICE DATE: 2-27-85 RECTIFICATION BY: AELYTEK, INC. 84031

DATE
DESIGN
DRAWN
CHECKED

Santa Clara Valley Water District

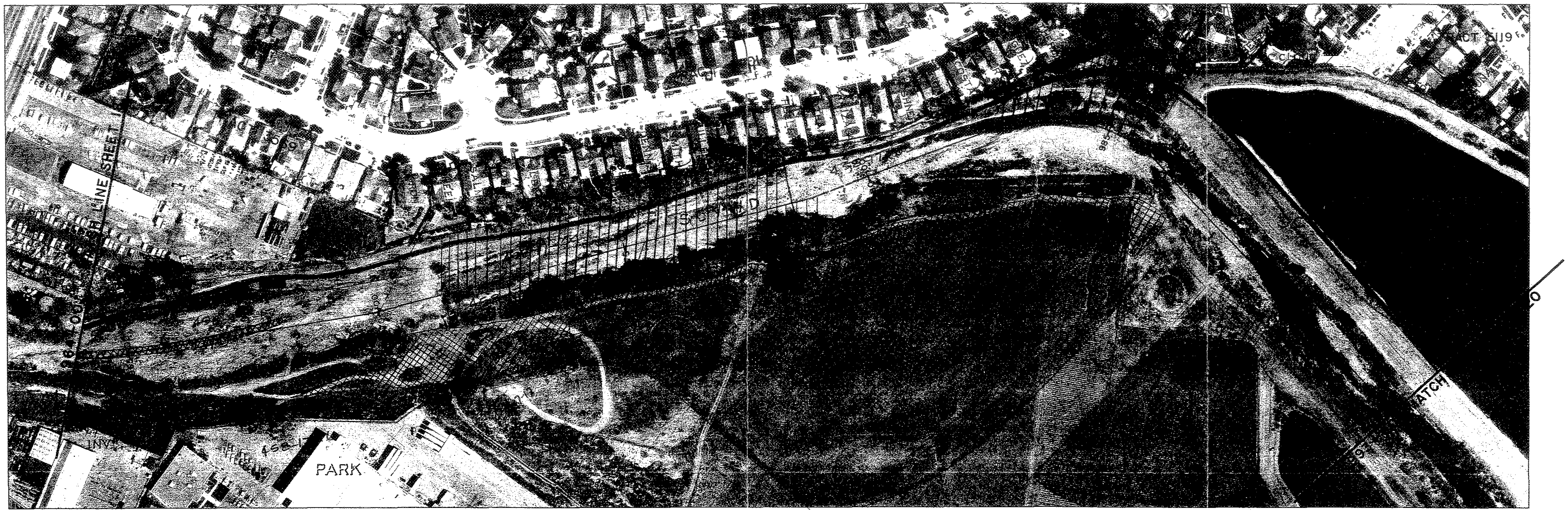
ENGINEERING CERTIFICATION
PROJECT ENGINEER R.C.E. NO.
DIVISION ENGINEER R.C.E. NO.

ENGINEERING APPROVAL
HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE 1" = 100'

SHEET 18



▨ SCVWD PROPERTY

▨ OTHER PROPERTIES TO BE ACQUIRED

—— PROPOSED R/W

EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT NEEDED ONLY

▩ PROPOSED MITIGATION AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.
	12		

REFERENCE INFORMATION AND NOTES:

PHOTOGRAPHY BY: AIR FLIGHT SERVICE
DATE: 2-27-85
RECTIFICATION BY: AERTEC, INC. 84031

DATE
DESIGN
DRAWN
CHECKED

Santa Clara Valley Water District

ENGINEERING CERTIFICATION
PROJECT ENGINEER R.C.E. NO.
DIVISION ENGINEER R.C.E. NO.

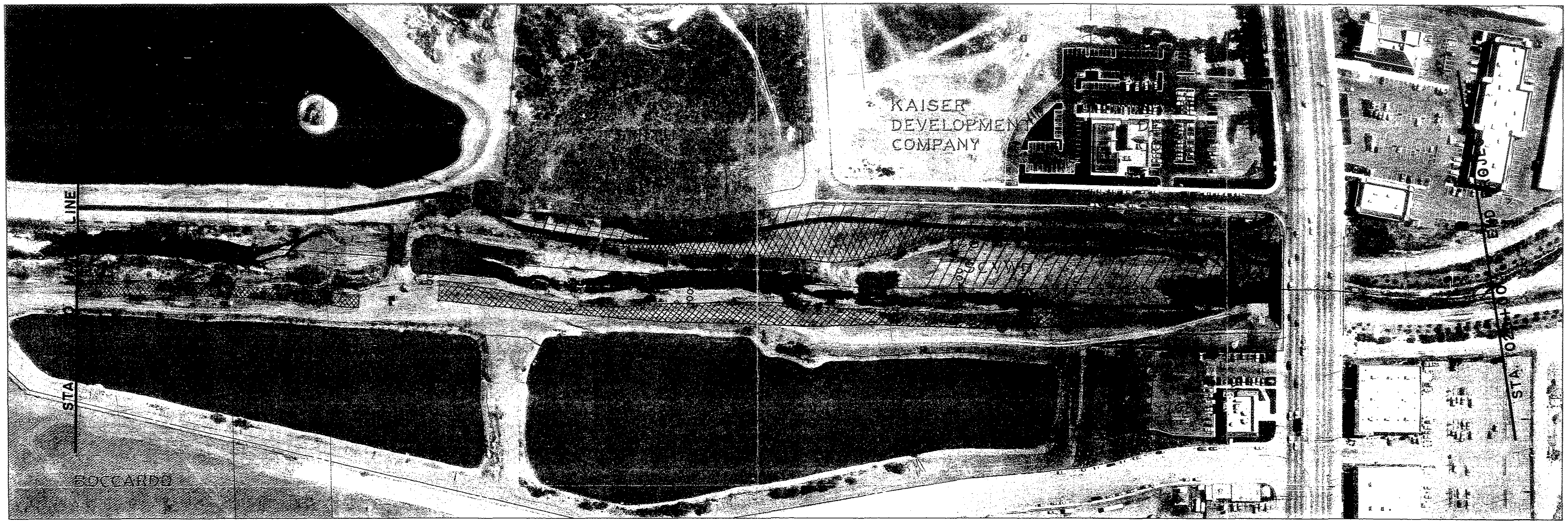
ENGINEERING APPROVAL
HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

GUADALUPE RIVER

SCALE
1" = 100'

SHEET
19

Ⓩ



▭ SCVWD PROPERTY

▒ OTHER PROPERTIES TO BE ACQUIRED

——— PROPOSED R/W

— — — EXISTING TOP-OF-BANK

▨ MAINTENANCE EASEMENT NEEDED ONLY

▩ PROPOSED MITIGATION AREA

▬ RECREATION TRAIL

REV	DESCRIPTION	DATE	APPR.

REFERENCE INFORMATION AND NOTES:
⊙224 EL 191.05

PHOTOGRAPHY BY: AIR FLIGHT SERVICE
DATE: 2-27-85
RECTIFICATION BY: AEROTEC, INC. 84031

DATE
DESIGN
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Santa Clara Valley Water District

ENGINEERING CERTIFICATION
PROJECT ENGINEER R.C.E. NO.
DIVISION ENGINEER R.C.E. NO.

ENGINEERING APPROVAL
HEAD, PROJECT DEVELOPMENT BRANCH R.C.E. NO.

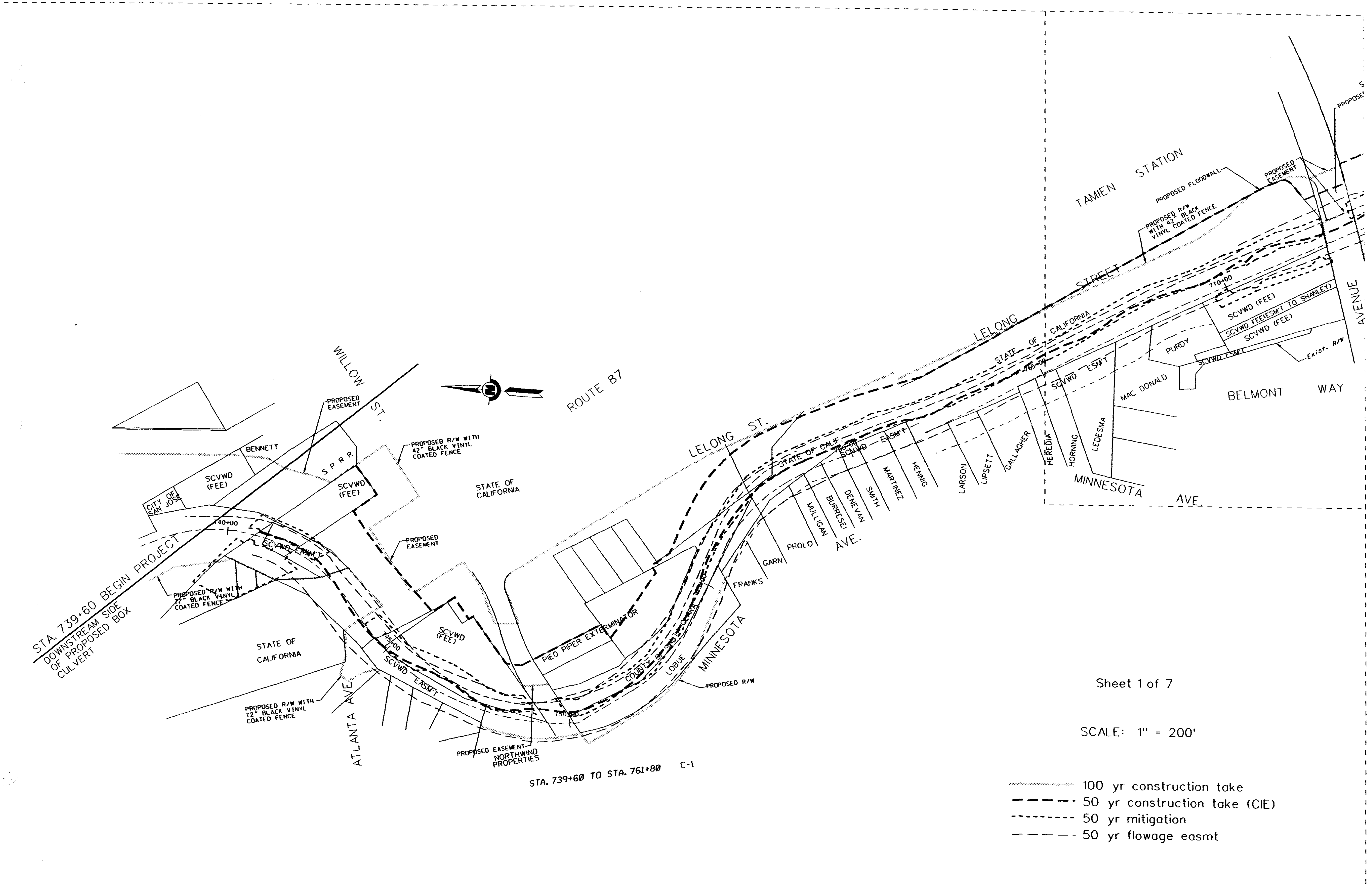
GUADALUPE RIVER

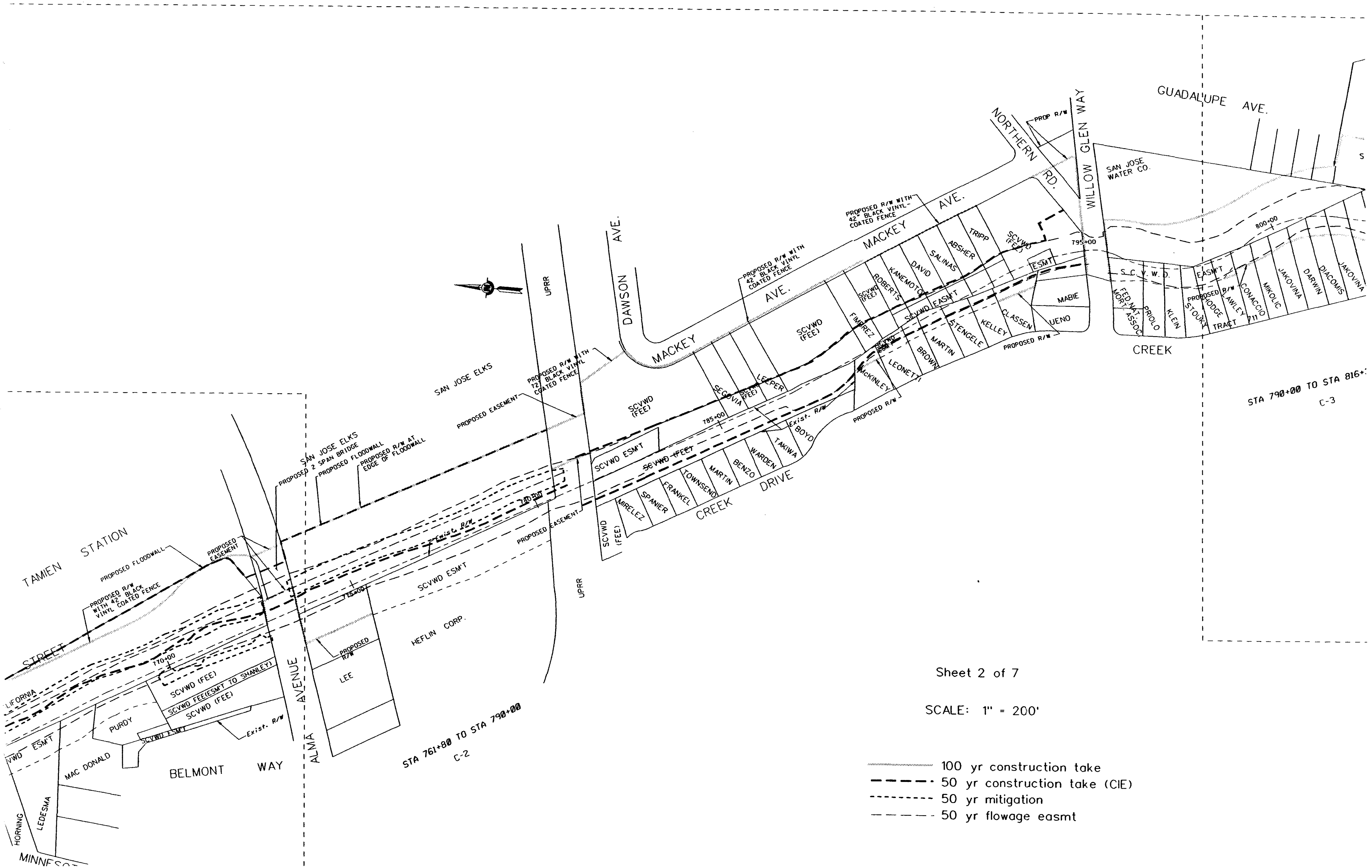
SCALE 1" = 100'

SHEET 20

EXHIBIT B

VALLEY VIEW PLAN (NED PLAN) PROJECT MAPS

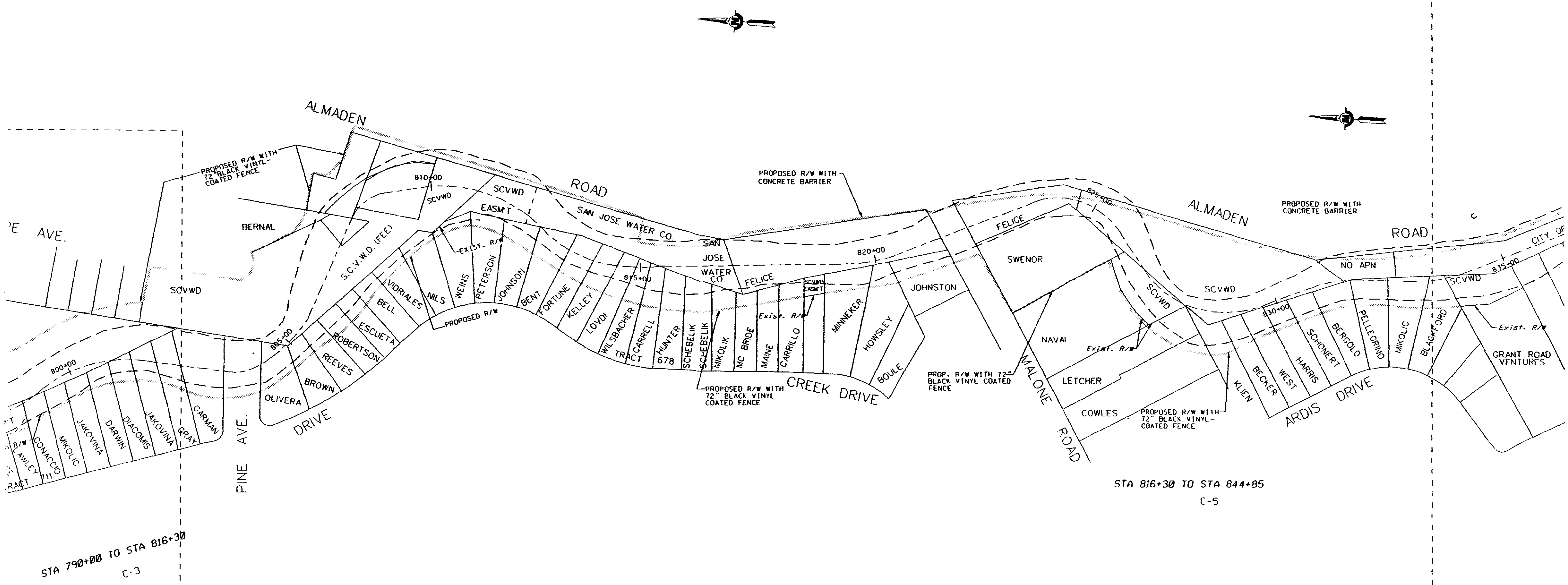

STA. 739+60 BEGIN PROJECT
DOWNSTREAM SIDE OF PROPOSED BOX CULVERT
CITY OF SAN JOSE
SCVWD (FEE)
BENNETT
PROPOSED EASEMENT
WILLOW ST.
SPRR
SCVWD (FEE)
PROPOSED R/W WITH 42" BLACK VINYL COATED FENCE
STATE OF CALIFORNIA
ROUTE 87
N
PROPOSED EASEMENT
740+00
SCVWD ESM'T
PROPOSED R/W WITH 72" BLACK VINYL COATED FENCE
STATE OF CALIFORNIA
745+00
SCVWD (FEE)
SCVWD EASM'T
PROPOSED R/W WITH 72" BLACK VINYL COATED FENCE
ATLANTA AVE.
PROPOSED EASEMENT
NORTHWIND PROPERTIES
PIED PIPER EXTERMINATOR
750+00
COUNTY OF SANTA CLARA
LOBUE
MINNESOTA
PROPOSED R/W
FRANKS
GARN
PROLO
AVE.
MULLIGAN
BURRESEI
DENEVAN
SMITH
MARTINEZ
HENNIG
LARSON
LIPSETT
GALLAGHER
HEREDIA
HORNING
LEDESMA
MINNESOTA AVE.
LELONG ST.
STATE OF CALIF
760+00
SCVWD EASM'T
LELONG STREET
STATE OF CALIFORNIA
765+00
SCVWD ESM'T
MAC DONALD
PURDY
SCVWD ESM'T
BELMONT WAY
TAMIEN STATION
PROPOSED FLOODWALL
PROPOSED R/W WITH 42" BLACK VINYL COATED FENCE
PROPOSED EASEMENT
770+00
SCVWD (FEE)
SCVWD FEE(ESM'T TO SHANLEY)
SCVWD (FEE)
Exist. R/W
AVENUE
PROPOSE
S
STA. 739+60 TO STA. 761+80
C-1
Sheet 1 of 7
SCALE: 1" = 200'
100 yr construction take
50 yr construction take (CIE)
50 yr mitigation
50 yr flowage easmt



Sheet 2 of 7

SCALE: 1" = 200'

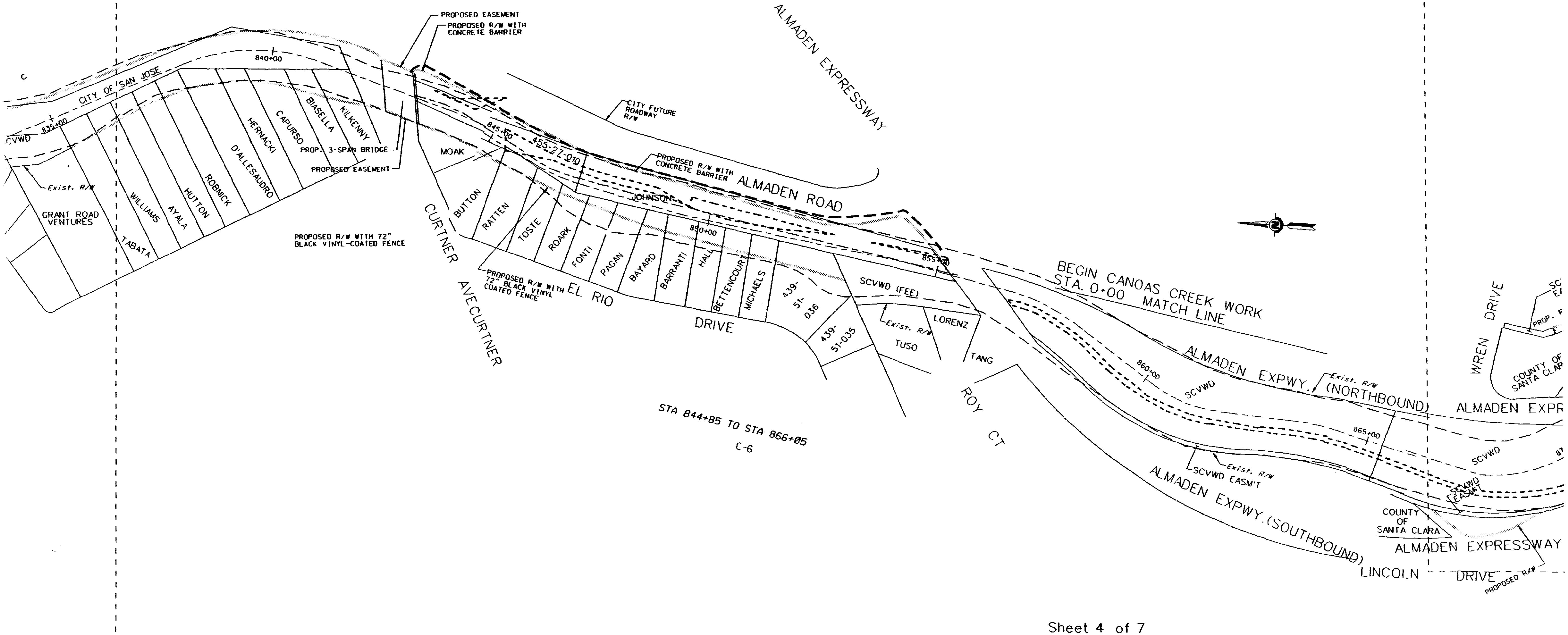
100 yr construction take
50 yr construction take (CIE)
50 yr mitigation
50 yr flowage easmt



Sheet 3 of 7

SCALE: 1" = 200'

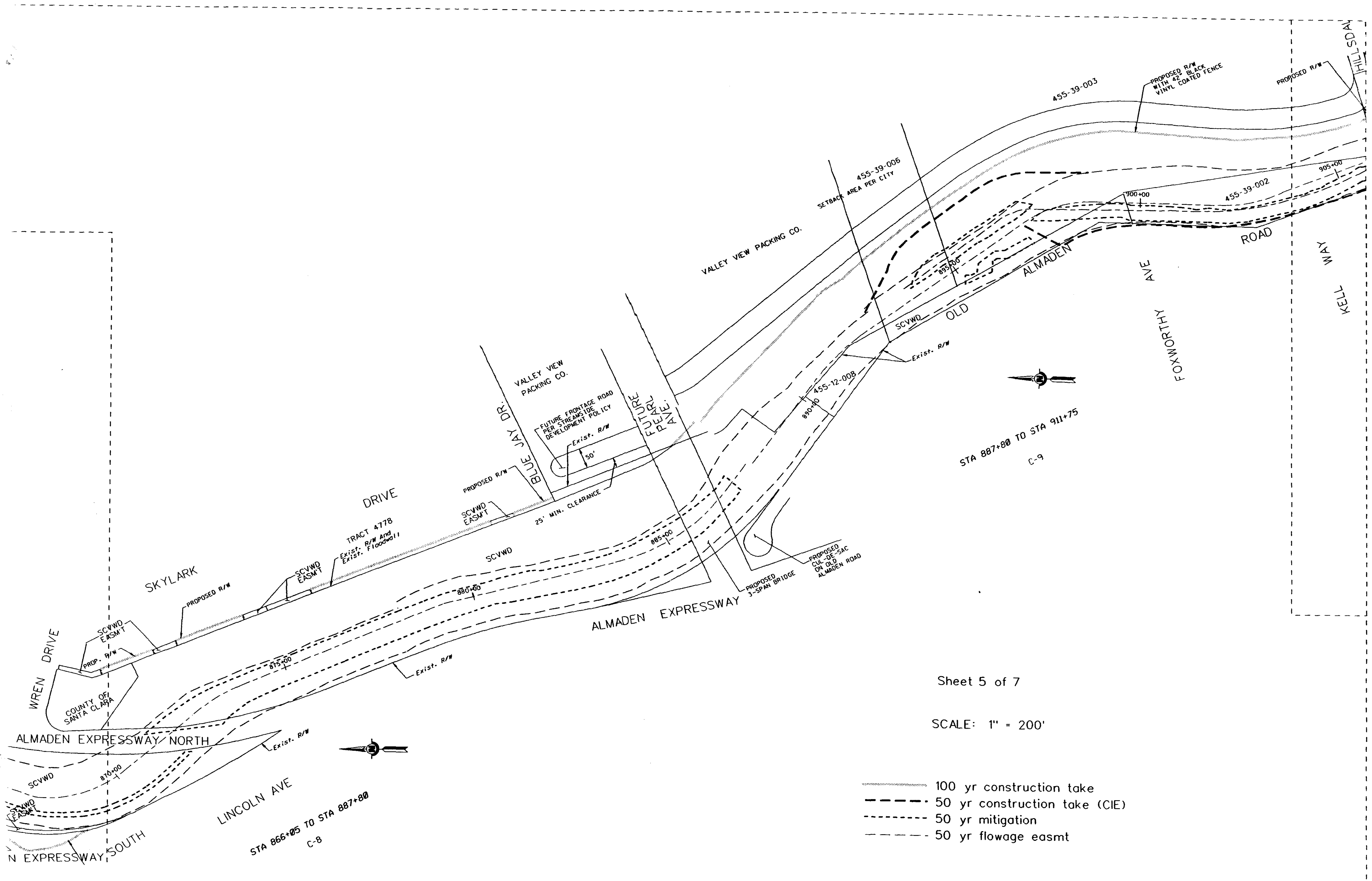
100 yr construction take
50 yr construction take (CIE)
50 yr mitigation
50 yr flowage easmt



Sheet 4 of 7

SCALE: 1" = 200'

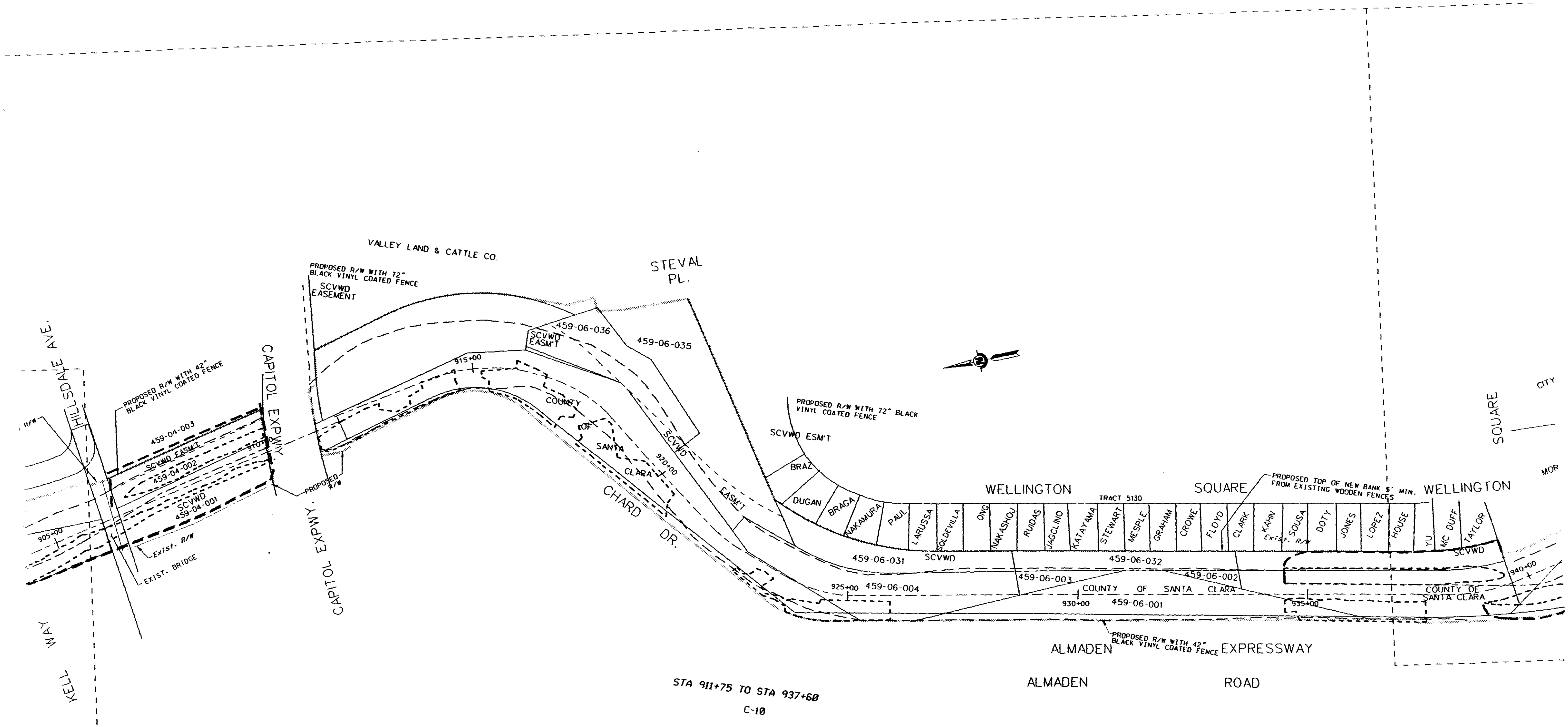
▨ 100 yr construction take
▬ ▬ 50 yr construction take (CIE)
- - - - 50 yr mitigation
— — — 50 yr flowage easmt



Sheet 5 of 7

SCALE: 1" = 200'

- 100 yr construction take
- 50 yr construction take (CIE)
- 50 yr mitigation
- 50 yr flowage easmt



STA 911+75 TO STA 937+60

C-10

Sheet 6 of 7

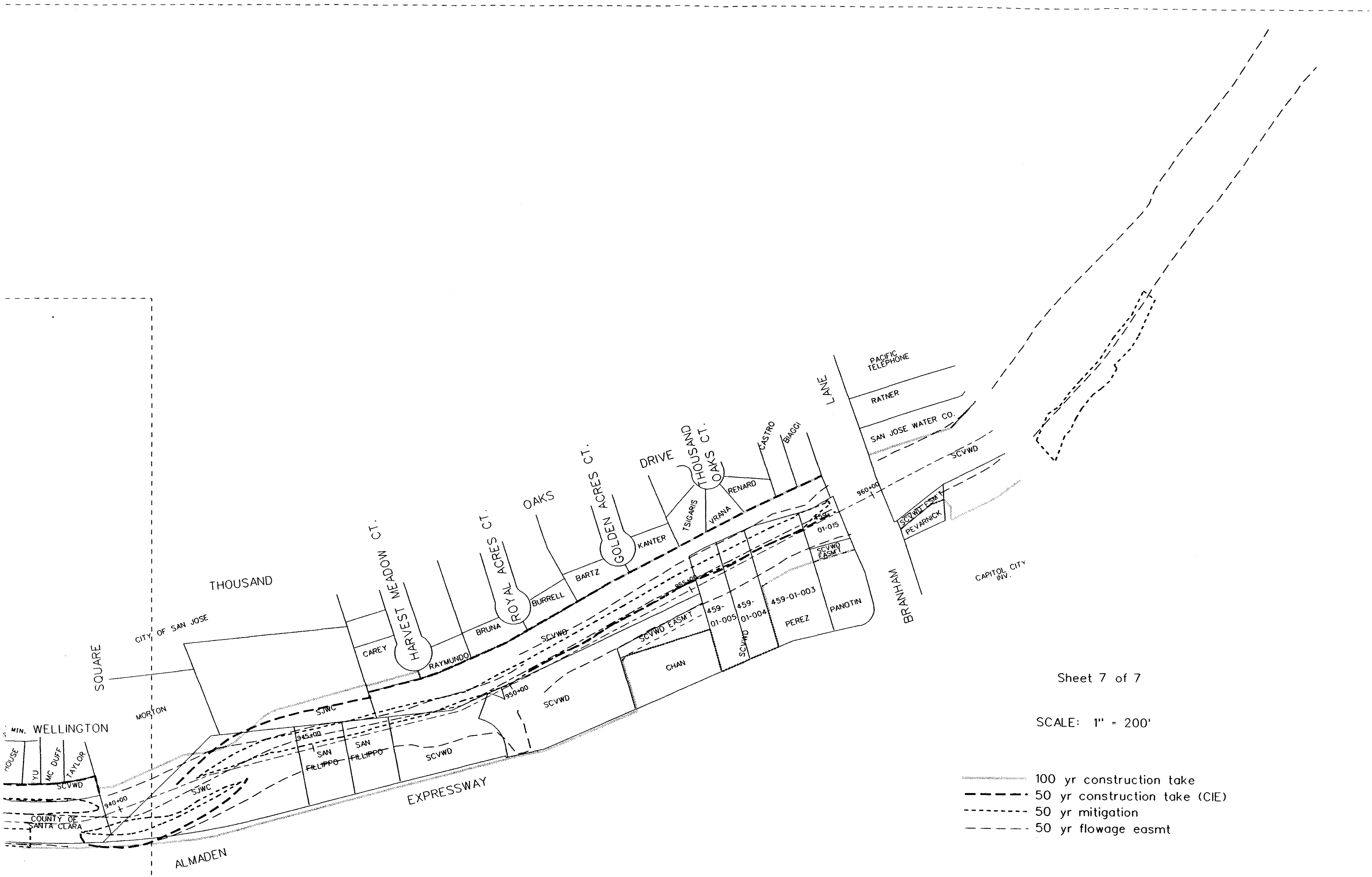
SCALE: 1" = 200'

▨ 100 yr construction take

▬ ▬ ▬ 50 yr construction take (CIE)

-------- 50 yr mitigation

— — — — 50 yr flowage easmt



Sheet 7 of 7

SCALE: 1" = 200'

ECONOMIC ANALYSIS

UPPER GUADALUPE RIVER
EVALUATION OF ECONOMIC BENEFITS

PURPOSE

This economic appendix has been prepared in order to determine the National Economic Development (NED) benefits associated with the flood damage reduction project for the upper Guadalupe River Project in San Jose, California. These benefits also serve to determine the "NED project", i.e., the project that maximizes net benefits or the difference between the total project costs and its beneficial contributions. Costs and benefits are expressed in average annual values using the FY 98 Federal Discount Rate of 7-1/8 percent, and a project life of 50 years. The project base year is 2004, and all benefits are expressed in 1995 dollars except where noted.

SCOPE

This analysis evaluated the various categories of flood protection benefits that are typically associated with flood control projects and presents the benefits, by category, determined to be actually affected by the project. As directed by the recent changes in flood evaluation procedures, EC-1105-205 (1994), a risk-based analysis (RBA) was initiated as part of the flood damage estimation procedure.

As part of the Corps refinement of evaluative procedures, the terminology related to project levels of protection and flood events has also been modified. For example, what was previously referred to as a 100-year event, is now more accurately described as a 1% chance occurrence event; a 50-year event is now referred to as a 2% chance occurrence event, etc. Using this percentage-based approach promotes a more precise understanding of how probable a flood event actually is. For example, multiple 50-year events could occur over a relatively short period of time. Using a percentage-based terminology explicitly addresses the idea that the probability of a flood events occurrence is the key metric. However, for ease of presentation year-event terminology may occasionally be used; see Table 1.

Table 1
Change in Terminology

Previous Terminology	New Terminology
20-year event	5% annual chance of occurring
50-year event	2% annual chance of occurring
100-year event	1% annual chance of occurring
500-year event	0.2% annual chance of occurring

STUDY AREA

The economic study area is located in San Jose, California in Santa Clara County. Santa Clara County is south of San Francisco Bay, bounded by Alameda to the north, Santa Cruz to the south and San Mateo County to the west. The study area has been broken down into five areas of economic analysis and consists of 2710 acres in the Upper Guadalupe River 100-year floodplain and 3260 acres in the 500-year floodplain. Table 2 shows the number of structures by land use type and by floodplain. Table 3 shows the number of structures in the 100-year floodplain by economic area and Table 4 presents the total value of property (structure and contents) in the floodplain ($3,358,705,000).

Table 2
Upper Guadalupe River Flood Protection Study
Existing Number of Structures in Floodplain

Land Use Type	20-Year [5%]	50-Year [2%]	100-Year [1%]	500-Year [.2%]
Single Family Residential	2,693	3,879	6,097	7,189
Multi-Family Residential	204	543	660	703
Mobile Homes	0	22	170	170
Commercial/Industrial/ Public	42	429	575	659
Total	2,939	4,873	7,502	8,721

Source: Santa Clara Valley Water District.

Table 3	
Economic Area	Number of Structures
Area 1	1,734
Area 2	3,050
Area 3	987
Area 4	829
Area 5	902
Total	7,502

Source: San Francisco District, Economics Branch

Table 4
Total Value of Damageable Property
500-Year Floodplain

Land Use Category	Structure Value	Content Value
Single Family Residential	$1,089,441,000	$544,720,000
Multi-Family Residential	$331,917,000	$165,958,000
Mobile Homes	$3,695,000	$1,848,000
Commercial/Industrial/ Public	$579,646,000	$641,480,000
Total	$2,004,699,000	$1,354,006,000

Source: The Santa Clara County Assessors Rolls (1995-1996).
Price level = October 1995

ALTERNATIVES

Three alternatives are currently being evaluated to alleviate the flooding problems for the upper Guadalupe River area. The Valley View and Willow Glen Plans consist of channel widening in the form of structural improvements to the existing upper Guadalupe River. The downstream end of the project would tie into the downtown Guadalupe River Project at Interstate Highway 280 (I-280). The Valley View Plan is being considered for protection against the 2% probability exceedance flood, and the Willow Glen Plan is being considered for a 5% probability exceedance flood.

The Bypass Channel Plan is similar to the Valley View and Willow Glen Plans except that a bypass channel would be constructed parallel to the east bank of the existing natural channel from I-280 upstream to Willow Glen Way. The Bypass Channel Plan is being considered for protection against the 1% probability exceedance flood.

MAJOR FINDINGS

The NED benefits for the Bypass Channel Plan, using the future hydrologic flow regime, are $24,924,000. The Bypass Channel plan is designed to offer protection against the 100-Year [1% probability exceedance] flood. Flood reduction benefits provide the greatest proportion of these benefits at $20,411,000. The remaining benefits accrue from: savings in the administration of the Federal Flood Insurance Program (FFIP) ($208,000), the reduction of traffic disruptions ($179,000), savings in current maintenance costs ($210,000), recreation benefits ($3,018,000), advance bridge replacement savings ($570,000) and savings in emergency flood costs to the Federal Emergency Management Agency (FEMA) ($328,000).

The Valley View Plan provides protection, using the future hydrologic flow regime, from the 50-Year [2% probability exceedance] flood and has total benefits of $18,313,000. The benefits consist of the following: flood damage reduction benefits ($17,343,000), savings in the administration of the FFIP ($65,000), the reduction of traffic disruptions ($136,000), savings in the current cost of maintenance ($126,000), advance bridge replacement benefits ($350,000) and savings in the cost of emergency flood relief ($293,000). The Valley View Plan has no benefits for recreation.

The Willow Glen Plan provides protection, using the future hydrologic flow regime, from the 20-Year [5% probability exceedance] flood and has total benefits of $15,134,000. The benefits consist of the following: flood damage reduction benefits ($14,460,000), savings in the administration of the FFIP ($36,000), the reduction of traffic disruptions ($74,000), savings in the current cost of maintenance ($126,000), advance bridge replacement benefits ($156,000) and savings in the cost of emergency flood relief ($282,000). The Willow Glen Plan also has no benefits for recreation.

Historical Storm Damages

Modern historical information regarding storm damages within the Guadalupe River basin can be found in flood damage reports published by Santa Clara County and by the Corps of Engineers. These reports summarize storm events and associated damages for Santa Clara County and for the Guadalupe River basin. However, no specific damages are available for the particular portion of the river within the study area.

Table 5 below presents a summary of the major historical flood events on the Guadalupe River system and basin-wide or county-wide damages in terms of historical dollars. It is important to note that this summary is a macro view of damages for Santa Clara County as a whole and is not specifically representative of the area under study. A short description of the events is presented after the table for additional information

TABLE 5
Historical Flooding On The Guadalupe River In Santa Clara County

HISTORICAL FLOOD (1)	MAGNITUDE OF EVENT		DAMAGE ESTIMATE
	Flow (cfs)	Freq. (yrs)	Historical $
December 1955	-	-	$753,500 (2)
April 1958	9,150 @ St. John Street	5	$1,348,000 (2)
March 1982	5,642 @ Almaden Exp.	8	$14,740,000 (3)
January 1983*	8,400 @ Almaden Exp.	18	Not Available
January 1995*	8,470 @ Almaden Exp.	14	$3,000,000 (3)
March 1995*	5,590 @ Almaden Exp.	6	$6,000,000 (3)

(1) Anecdotal evidence also suggests flooding in 1862, 1867, 1895, 1911, 1917, 1950 and 1963
(2) Damages along entire mainstem, including areas beyond study area boundaries
(3) Countywide damages which may include areas beyond Guadalupe basin.
*Santa Clara County declared a National Disaster Area

April 1958: One of the highest discharges on record was produced by the flood of 1958 when floodwaters overbanked in downtown San Jose and covered a two square block area to depths of up to four feet.

March 1982: The river's east bank was overtopped inundating about 15 acres between the Union Pacific Railroad crossing and W. Virginia Street. The under crossing of the Southern Pacific

Railroad at Willow Street and Alma Street filled with floodwaters to a depth of ten feet. The San Jose Elks lodge reported one foot of water inside its building on Alma Street. Homes on McLellan Avenue were evacuated, and twenty homes and five businesses reported flood damages in this vicinity. No specific damages are available for this area. Countywide damages were sustained by 480 homes ($1,750,000) and 78 businesses ($1,750,000). Total private damages were $3,670,000 and public damages were $6,430,000.

January 1983: During the January 1983 storm, flows overtopped the banks of the Guadalupe River between Alma Street and the Western Pacific Railroad bridge. Widespread flooding resulted over approximately 15 acres extending from the Elks Lodge on Alma Street to West Virginia Street and east of the river to the Southern Pacific Railroad tracks and McLellan Avenue. The San Jose Elks lodge reported one foot of water inside its building on Alma Street. Approximately ten feet of water collected at the Willow Street undercrossing of the SPRR Bridge. No specific damages are available for this area.

January 1995: During this storm, the Guadalupe River overtopped its banks at River Street; near Alma Avenue; and at Virginia Street, where water flowed onto Highway 87. Depths of 15 feet were reached between Virginia Street and Alma Avenue. Highway 87 and the light rail tracks were submerged. The cleanup costs were estimated to be $100,000. Highway 87 remained closed in the southbound direction for two days, and the northbound lanes were closed for three days. Ross Creek overtopped its banks at Cherry Avenue and Jarvis Avenue. Canoas Creek overtopped its banks at Redbird Drive, Kingfisher Drive, Calero Avenue, and Blossom Avenue.

March 1995: During the March 1995 storm, the river overtopped its banks between Alma Street and the Union Pacific Railroad bridge. The Elks' Lodge was flooded. Willow Street and properties on McLellan Avenue, Harliss Avenue, Belmont Way, and River Street were flooded. Homes along Virginia and St. John Streets were evacuated. Highway 87 was also flooded. The areas inundated within the study area during the 1995 storms are shown in the Hydrology and Hydraulics Appendix.

ER 1105-2-100 states that "the basis for the determination of existing damages is losses actually sustained in historical floods;" It goes on to state that "historical data are often incomplete...therefore, data on historical flood losses should be carefully scrutinized and supplemented by appraisals, use of area depth-damage curves and an inventory of capital investment within the floodplain. Further, estimates of damages under existing conditions should be completed for floods of magnitude that have not historically occurred."

In accordance with the guidance in ER 1105-2-100, an examination of historical damages was conducted. All of the information listed in the table above was reviewed in order to help develop the existing damages. Unfortunately, the aggregate nature of the reference material limits its usefulness in the compilation of existing damages. The damage information in the reference material, instead of being exclusive to the study area, is aggregated into damages sustained to the entire Guadalupe basin or for damages sustained to the county of Santa Clara as a whole. Consequently, the reference material is very broad; it was not designed to be, nor was it intended to be a comprehensive damage survey. These surveys did not specify a floodplain, an overall inventory, or even specify in any detail the source data that supports their damage estimate. The primary purpose of these damage reports was to document public damages in order to provide a basis for FEMA reimbursement of the county for countywide cleanup, and other flood related costs.

A good example of the of the county wide nature of the reference material, and it's inherent variability with regards to the study area, is the difference between the January 1995 event and the March of 1995 event. In the table above the January event is listed as having a cfs of 8470 at the Almaden gauging station with damages of approximately $3,000,000, while the March event at the same gauging station registers only 5590 cfs, and double the damage of the January events $3,000,000. The reason for this apparent contradiction is that the damages are county wide and while the March flood was only a 6 year event at the Almaden gauging station, which is in the immediate study area, at other places in the county, outside of the study area, it was larger; hence the variability in damages between the January and the March events.

The question of whether the existing damages, as calculated in the Flood Damage section, are compatible with the historic damages, is best addressed by a comparison of the existing damages to those of an historic event. Even though the comparison can only be considered in the broadest context, given the limited and fragmentary nature of the historic damages and their county wide scope, these damages will nonetheless, be useful in confirming the reasonableness of the damage estimate contained in the Flood Damage section.

Santa Clara County prepared a "Report On Flooding And Flood Related Damages" for the January 1995 and the March 1995 floods in Santa Clara County. Even though the report is a broad overview of the damages in the entire county, they can be used to provide a snapshot view of damages. The March 1995 event was rated as a 6-year event at the Almaden gauging station, which is in the study area. According to the county report, 195 private properties were directly damaged by the flood with damages of over $3,300,000. This means the approximate damage per directly effected property in the March, 1995 flood was approximately $17,000. Because this was a relatively small event at our gauging station (the Almaden station), a comparison with the existing damages 20-year flood and 20 year damages would be the most informative. The number of structures in the 20 year floodplain is approximately 3,000 as seen in Table 2, if we compare this with the amount of projected damages for a 20 year event ($43,887,000 as seen in Table 7) we get approximately $14,600 in damages per property. From this comparison, it is clear that the existing damages are compatible with the historic damages, especially given the non-specific and fragmentary nature of the historic damages.

More detailed information on the derivation of the RBA of existing damages is presented in the Flood Damages section.

PROJECT BENEFITS

The positive effects resulting from a flood control project are the flood control project benefits. For this project, these benefit categories include flood damage reduction benefits, savings in flood insurance administrative costs, savings in current maintenance costs, recreation benefits, advance bridge replacement benefits and savings in emergency flood costs.

FLOOD DAMAGES

Approximately 84 percent of the total benefits are flood damage reduction benefits. Flood damage reduction benefits are the difference between the average annual equivalent (AAE) damages without the project and the residual AAE damages with the project.

The determination of flood damage is a 2-step process utilizing RBA. EC-1105-2-205 provides guidelines for conducting RBA. According to the EC, RBA is defined as an approach to evaluation and decision making that explicitly, and to the extent practical, analytically incorporates consideration of risk and uncertainty. It is recognized that the "true" values of key planning and design variables and parameters are frequently not known with certainty and can take on a range of values. One can describe, however, the likelihood of a parameter taking on a particular value by a probability distribution. The probability distribution is described by its mean and variance for a normal distribution, or minimum, maximum, and most likely for a triangular distribution. The approach combines the underlying risk and uncertainty information so that the engineering and economic performance and associated reliability of a project can be expressed in terms of probability distributions.

The first step in computing damages is a RBA utilizing a computerized damage calculation model that explicitly incorporates risk factors into the damage model. This damage calculation model uses a representative sample of structures in the floodplain to extrapolate the damages in the study area for a specific event. In this case, the damages were calculated for eight specific events. The model calculated the damages associated with present and future hydraulic conditions for the 20-, 50-, 100-, and the 500-year events

This phase of damage calculation is designed to incorporate the inherent variability of measurement risk and uncertainty into the model. At this point the variability of the first floor measurements, structure values and other variables are inputted in to the model as distributions. This allows the model to run the damages for the eight specific events described above and create an aggregate stage damage curve. This aggregate stage damage curve is then used as the base data in the Corps/HEC risk program.

The damage model uses Lotus 1-2-3 and the @RISK computer program to determine baseline damages and to incorporate uncertainty into the model. The model utilizes a spreadsheet (Developed by USAED, San Francisco District) environment with the @RISK computer program add in to calculate event damages. Event damages are computed by combining the economic inventory data, the field work data, and the hydraulic data. The base data contained in the model is: a structure population sample representing all of the damageable property in the floodplain; the elevations associated with each of the sample properties; value data and water surface elevations associated with the flood event under consideration.

The spreadsheet model is organized into Columns and Rows. Each "row" of data represents one sampled property and each "column" of data contains key variables and formulas linked to the sample property. Column entries consist of the improvement and content value of the property, the property elevation data, the property type, the number of properties (the expansion factor) represented by the sample and a number of other factors. In order to introduce uncertainty into the model key variables are represented by probability distributions. These distributions allow the key variables to take on a range of values which explicitly incorporate risk and uncertainty into the model.

The damage models calculations consider the type of structure, the value of the structure, the value of the contents, and the depths of flooding associated with each of the various flood events evaluated. The value of a structures "contents" was not independently determined. Instead, ratios of content value to structure value were applied by land use and structure type. These ratios are based on recent property surveys conducted by the Corps (San Francisco District) within the San Francisco Bay Area. The ratio for commercial properties was 100 percent and the ratio for residential properties was 50 percent. They represent the best available information to apply to the various types of residential and non-residential properties. The content values also incorporate risk and uncertainty because they are a function of the improvement value distributions

The program then estimates economic damages by arraying and comparing the economic data, which has been inputted into the program as @RISK distributions, to the hydraulic data, which is deterministic. Essentially the program derives damages in the floodplain by comparing the water surface elevations associated with a particular flood event to the elevation of structures in the floodplain. It uses the depth of flooding associated with a specific flood event and matches that level of flooding to a depth damage curve that is appropriate for the structure being evaluated. The estimated damages for the sample properties are then expanded based on the samples expansion factor. All of these calculations are done in an RBA environment.

Standard depth damage curves were used to project the damages to residential and commercial structures and contents. These structure and content depth damage curves relate the level of inundation to the percent of damage incurred by a structure. They are the result of empirical studies by the Corps -San Francisco District- and are considered to be appropriate for the area.

The @RISK program calculates damages by running thousands of @RISK iterations for each event under consideration. Each time the damage model runs an @RISK iteration, it draws values from the variables that have distributions associated with them and then uses these results to recalculate economic damages. Each time @RISK runs an iteration it generates and then stores the results of that run. After 5000 iterations of an event, the most likely value is determined by averaging all of the different damages generated for that event. This process determines the without project damages for the event under consideration. The resulting damage estimates for each of these eight events constitutes the foundation of the aggregate stage damage (ASD) curve. The @RISK damages for each event are displayed in Table 6.

Table 6
Aggregate Stage Damage Curves For Areas 1 & 2

	Economic Area 1			Economic Area 2		
Flood Event	cfs	Elev	Damage	cfs	Elev.*	Damage
Zero Damage	6.5 Year Event			5.5 Year Event		
Present 20-Year (5%)	8,800	92.0	$8,724,000	1,050	135.4	$32,020,000
Future 20-Year (5%)	9,000	92.3	$9,741,000	1,550	135.8	$45,147,000
Present 50-Year (2%)	11,000	93.5	$12,319,000	1,450	135.8	$41,059,000
Future 50-Year (2%)	11,200	93.8	$13,232,000	1,950	136.4	$66,526,000
Present 100-Year (1%)	13,200	94.0	$18,336,000	1,700	136.0	$55,547,000
Future 100-Year (1%)	14,600	94.3	$20,189,000	2,250	136.7	$83,732,000
Present 500-Year (0.2%)	18,000	95.0	$27,334,000	2,500	136.7	$100,932,000
Future 500-Year (0.2%)	21,800	95.3	$39,978,000	3,000	137.6	$134,515,000

Source: San Francisco District, Economics Branch. Price level = October 1995

Table 6 Continued
Aggregate Stage Damage Curves For Areas 3 & 4

	Economic Area 3			Economic Area 4		
Flood Event	Cfs	Elev.	Damage	cfs	Elev.	Damage
Zero Damage	7.14 Year Event			9 Year Event		
Present 20-Year (5%)	8,800	Sheetflow	$883,000	2,500	127.5	$2,260,000
Future 20-Year (5%)	9,000	Sheetflow	$895,000	2,500	127.8	$3,318,000
Present 50-Year (2%)	11,000	97.0	$8,207,000	2,900	131	$48,948,000
Future 50-Year (2%)	11,200	97.3	$9,529,000	3,000	131.3	$63,013,000
Present 100-Year (1%)	13,200	98.0	$10,063,000	3,200	133	$96,671,000
Future 100-Year (1%)	14,600	98.5	$13,104,000	3,300	133.3	$132,440,000
Present 500-Year (0.2%)	18,000	99.0	$23,966,000	3,500	134	$191,295,000
Future 500-Year (0.2%)	21,800	99.5	$30,949,000	3,600	134.3	$263,303,000

Source: San Francisco District, Economics Branch. Price level = October 1995

Table 6 Continued
Aggregate Stage Damage Curves For Area 5

	Economic Area 5		
	cfs	Elev.*	Damage
Zero Damage	37 Year Event		
Present 20-Year (5%)	6,300		$0
Future 20-Year (5%)	6,300		$0
Present 50-Year (2%)	9,100		$0
Future 50-Year (2%)	9,100		$0
Present 100-Year (1%)	11,400	173	$29,217,000
Future 100-Year (1%)	11,400	173	$30,986,000
Present 500-Year (0.2%)	17,000	174	$55,435,000
Future 500-Year (0.2%)	17,800	174	$55,805,000

Source: San Francisco District, Economics Branch
Price level = October 1995
* Out of bank measurements. These elevations represent reference elevations.

Tables 7 and 8 display present and future aggregate event damages by category. The damages under the present hydrologic flow regime range from approximately $44,000,000 for the 20-year event to approximately $400,000,000 for the 500-year event. The damages under the future hydrologic flow regime range from approximately $59,000,000 for the 20-year event to approximately $524,000,000 for the 500-year event.

Table 7
Total Without Project (Present) Damages by Flood Event

	Total of All Economic Areas			
	20-Year[5%]	50-Year[2%]	100-Year[1%]	500-Year[.2%]
Structure	$28,138,000	$52,569,000	$92,021,000	$172,578,000
Content	$12,624,000	$43,942,000	$95,385,000	$183,628,000
Automobile	$3,125,000	$14,022,000	$22,428,000	$42,756,000
Total -->	$43,887,000	$110,533,000	$209,834,000	$398,962,000

Source: Damage program, Economics Branch, San Francisco District.
Price level = October 1995.

Table 8
Total Without Project (Future) Damages By Flood Event

Total of All Economic Areas				
	20-Year[5%]	50-Year[2%]	100-Year[1%]	500-Year[.2%]
Structure	$35,672,000	$69,547,000	$117,682,000	$132,604,000
Content	$18,351,000	$63,594,000	$132,932,000	$327,168,000
Automobile	$5,078,000	$19,159,000	$29,837,000	$64,778,000
Total -->	$59,101,000	$152,300,000	$280,451,000	$524,550,000

Source: Damage program, Economics Branch, San Francisco District
Price level = October 1995

AVERAGE ANNUAL DAMAGES

For damage estimation purposes, the floodplain has been separated into five economic areas. The basis for this separation was that different areas of the floodplain were subject to different non-damaging frequencies or, where levees currently exist, probable failure points (PFP) and probable non-failure points (PNP). The upstream end of the floodplain is currently benefiting from higher levels of protection than the downstream end. These factors were considered when calculating the average annual damage figures.

As mentioned above the event damages in table 6 are used to construct an ASD curve. The economic variables are then combined with hydraulic and hydrologic variables to define the ASD. This data is then inputted into the Corps/HEC developed Risk Damage Model for the purpose of generating AAE damages. The H&H variables use the same basic format as the economic variables in that the floodplain is divided into 5 sub-areas, with the following assumptions:

1. A channel cross-section discharge-rating curve is developed for each sub-area at its unique flood source for each condition or project to be analyzed.

2. The water surface stage uncertainty is based on the uncertainties of the Manning's "n" value and the uncertainty of sedimentation.

Thus, the data input into the risk model includes:

a.) Discharge-Frequency Data; mean, skew, years of record.
b.) Aggregate Stage Damage Curve.
c.) Stage Discharge Curve.
d.) Uncertainty of Stage.

The results of the model are the AAE damages, and their distributions, i.e. standard deviations. Ten thousand iterations are performed similar to the process in Step 1 for each Area. First, it is run under without project conditions, then with the various project alternatives, i.e. 5%, 2%, and 1%. At this point all the different AAE damages generated are compared.

Table 9 presents the AAE damages for present hydrologic conditions and Table 10 presents the AAE damages for the future hydrologic conditions. The future hydrologic conditions reflect the anticipated upstream buildout of the watershed and the concomitant increase in water surface elevations. The economic damages for the future conditions are based on the assumed future hydrologic conditions. These future hydrologic conditions take into consideration the changing nature of the urban landscape and the increased density of urbanization in the watershed. The derivation of the future hydrologic conditions use data from the San Jose General Plan and have been approved as appropriate for use in this study. This increase in flood levels varies by area from 0 to 0.5 feet.

Table 9*
AAE Damages for Study Alternatives
for Present Conditions ($000)

Economic Area	W/O Project	W/ Project Residuals		
		Willow Glen	Valley View	Bypass Channel
Area 1	2,031	539	309	89
Area 2	11,175	2,675	1,593	534
Area 3	1,743	263	154	105
Area 4	7,313	6,000	5,100	4,200
Area 5	1,223	1,223	1,004	403
Total	23,485	10,700	8,160	5,331

*Represents Annual Flood Damages For Present Hydraulic Conditions using Risk Analysis.
Price level = October 1995

Table 10*
Economic Damages for Study Alternatives
For Future Conditions ($000)

Economic Area	W/O Project	W/ Project Residuals		
		Willow Glen	Valley View	Bypass Channel
Area 1	2,310	634	364	108
Area 2	11,763	2,816	1,617	562
Area 3	2,051	309	181	123
Area 4	7,313	6,000	5,100	4,200
Area 5	1,287	1,287	1,057	424
Total	24,724	11,046	8,319	5,417

*Represents Annual Flood Damages Future Hydraulic Conditions using Risk Analysis.
Price level = October 1995

Projects providing varying levels of protection were analyzed. The Bypass Channel Plan is effective in eliminating all flooding for 20-, 50-, and 100-year events, except in Economic Area 4. While not fully analyzed, even a 500-year project would receive some residual flooding. The Valley View Plan is effective in eliminating flooding with Willow Glen and 50-year events but still receives some residual flooding in 100-year and 500-year events. The Willow Glen Plan is effective in eliminating flooding for a 20-year event but still receives residual flooding with 50-, 100-, and 500-year events. Again, as with the Bypass Channel Plan some residual flooding is anticipated in Area 4 for the Willow Glen and Valley View Plans.

FLOOD CONTROL BENEFITS

The flood control benefits, derived for the Bypass Channel Plan [1%] are $19,307,000, for the Valley View Plan [2%], they are $16,405,000, and for the Willow Glen [5%] plan, they are $13,678,000. These benefits are based on future flows and the standard "With" project vs. the "Without" project principle. The results are shown by economic area in Table 12. Benefits Based on present flows are presented in Table 11.

Table 11
Economic Benefits for Study Alternatives Under Present Hydraulic Conditions ($000)

Economic Area	Project Benefits		
	Willow Glen	Valley View	Bypass Channel
Area 1	$1,492	$1,722	$1,942
Area 2	$8,500	$9,582	$10,641
Area 3	$1,480	$1,589	$1,638
Area 4	$1,313	$2,213	$3,113
Area 5	0	$219	$820
Total	$12,785	$15,324	$18,154

Source: @RISK analysis provided by Hydrology and Coastal Section.
Price level = October 1995

The economic benefits for the future conditions are based on the assumed future hydrologic conditions. These future hydrologic conditions take into consideration the changing nature of the urban landscape and the increased density of urbanization in the watershed. The derivation of the future hydrologic conditions use data from the San Jose General Plan and have been approved as appropriate for use in this study.

Table 12
Economic Benefits for Study Alternatives Under Future Hydraulic Conditions ($000)

Economic Area	Project Benefits		
	Willow Glen	Valley View	Bypass Channel
Area 1	$1,676	$1,946	$2,202
Area 2	$8,947	$10,146	$1,1201
Area 3	$1,742	$1,870	$1,928
Area 4	$1,313	$2,213	$3,113
Area 5	0	$230	$863
Total	$13,678	$16,405	$19,307

Source: @RISK analysis provided by Hydrology and Coastal Section. Price level = October 1995

FEDERAL FLOOD INSURANCE ADMINISTRATIVE COSTS

In 1973 the Flood Disaster Protection Act required that residence in flood prone areas purchase flood insurance from the Federal Flood Insurance Program (FFIP). The FFIP is administered by FEMA and provides insurance to those flood prone areas. There is a national cost associated with the administration of FFIP. With a project in place, there will be a savings of this cost which can be captured as a project benefit.

The benefit of the reduction in flood insurance costs would accrue to the increment of structures in the zone between the boundary of the without project 100-year floodplain and the boundary of the with project 100-year residual floodplain. Those structures that are still contained within the boundaries of the proposed with project residual 100 year floodplain would not be eligible to claim The flood insurance cost reduction benefit.

The actual calculation of this benefit is derived by multiplying the number of structures that have been removed from the 100-year floodplain (by implementation of the proposed with project plan) by the estimated FFIP Participation Rate. This total is then multiplied by the cost of administering an FFIP policy to give the total benefit per proposed plan. The 1995 Fiscal Year National flood Insurance Program operating cost is $115 per policy and the estimated Participation Rate is 24 percent.

A conservative estimate of the FFIP Participation Rate was deemed to be 24 percent. The 24 percent Participation Rate was derived after consultation with both FEMA and the City of San Jose's Department of Public Works. According to the San Jose Department of Public Works biennial report, the city of San Jose has an estimated 29,000 structures within flood prone areas. FEMA was also contacted and they reported that, as of November of 1996, there were 7,060 flood insurance policies in the City of San Jose. Given the assumption that the flood insurance Participation Rate of the study area and the City are the same, the resulting rate would be 24 percent.

However, both the city of San Jose's Department of Public Works and FEMA believe that the Participation Rate will rise. San Jose's Department of Public Works believes that the Participation Rate will rise to seventy or eighty percent due to the Flood Insurance Reform Act of 1994. The act authorizes the Federal government to levy fines on lending institutions that provide loans to potential borrowers who do not purchase flood insurance. Federal agencies have been trained to incorporate fines into their audits (Office of Thrift Supervision, NCUA, etc.) Because of these developments, it is believed that a Participation Rate of closer to 50 percent may actually reflect current conditions. However, for the purposes of this report the more conservative confirmable estimate of a 24 percent Participation Rate will be reflected.

There are approximately 7,500 structures within the upper Guadalupe 100-year floodplain. After implementation of the Bypass Channel Plan there will be approximately 880 structures still contained within the 100-year residual floodplain. This means that approximately 6,620 structures will have been removed from the 100-year floodplain through implementation of the Bypass Channel Plan. Similarly, if the Valley View Plan were implemented 2,060 structures would be removed from the 100-year floodplain; implementation of the Willow Glen Plan would remove 1,148 structures from the 100-year floodplain. Table 13 illustrates the savings in flood insurance costs for the three alternatives.

Table 13
Savings in Flood Insurance Costs

Alternative	Structures Removed from .01 Floodplain	Participation Rate	Policy Cost	Benefit
Bypass Channel	6620	24 Percent	$115	$183,000
Valley View	2060	24 Percent	$115	$57,000
Willow Glen	1148	24 Percent	$115	$32,000

Price level = 1995

TRAVEL DISRUPTION COSTS

Under the "Without Project" condition significant travel delays are incurred by motorists during flood events. This traffic disruption analysis focuses on the difference in delays under the "Without project condition and the "With Project" condition. The analysis also calculates the extra mileage costs incurred through flood related travel disruptions and the cleanup costs of restoring traffic viability after an event. Using this data the Average Annual Benefit from the reduction in disruption costs, attributable to project implementation, was calculated.

Appendix 2 details the assumptions and the data used to derive the Average Annual Benefit attributable to project implementation. A summary table of the benefits is included below.

Table 14
Average Annual Traffic Disruption Reduction Benefit

	Willow Glen Plan	Valley View Plan	Bypass Channel Plan
Benefits	$74,000	$136,000	$179,000

SAVINGS IN CURRENT MAINTENANCE

From I-280 to the Canoas Creek confluence, the maintenance work was limited to emergency channel clearing and bank stabilization due to lack of right of way and maintenance access. The work consisted primarily of removal of blockages, such as downed trees, cleaning bridge pier noses of debris, and emergency erosion repair of failed or failing banks. As a result of erosive velocities in the channel, several large erosion repair projects have been constructed at various locations in the past 20 years. Based on historical records provided by the Santa Clara Valley Water District, the current annual cost of this maintenance is estimated to be $200,000.

All three of the proposed alternatives would take over the ongoing maintenance obligation of the existing river channel. The cost savings, and therefore the benefit, would be achieved by eliminating all or a proportion of the current maintenance expenditures through implementation of a project. The 1% plan effectively eliminates the need to provide maintenance to the current channel, resulting in a savings of $200,000. Implementation of the 2 percent plan or the 5 percent plan is expected to reduce the annual maintenance cost on the existing channel by 40 percent; thus, the savings would be $120,000.

The reduction in the maintenance cost of the current channel is a benefit attributable to the project. It must be realized that any new maintenance costs associated with the implementation of the authorized project are in fact costs, costs that will be recognized in the cost side of the project.

RECREATION BENEFITS

As part of a flood control project a service road will be developed that also can be used to serve recreational needs. A road will be developed adjacent to the creek primarily for maintenance purposes, but for levels of protection less than 1% it would not be a continuous road and would only be built in certain pockets along the river. Hence, no significant level of additional recreation benefits were considered for levels of protection less than 1%. However, the Bypass Channel plan provides a maintenance road that extends basically along the entire length of the river, and provides for extensive recreational opportunities. It would serve as a connector between downtown San Jose and residential areas. Furthermore, links to existing upland trails would provide both access to the river and to a continuous path. For modest added costs it can provide a very high recreational benefits and also provide connection to Los Gatos Creek Trail. The recreation derived is a natural outgrowth of the 1% protection plan and is essentially incidental, in providing jogging, walking and biking, utilizing the proposed project's service road. Such a trail is envisioned as part of the Bypass Channel Plan [1%]. The 5% and 2% chance level of protection plans have roads in only pockets and do not create a continuous path to accommodate the type of recreation cited above.

A Unit Day Value Method (UDVM) of analysis was used to develop the recreational benefits for this study. Upon completion of the analysis it was determined that the Bypass Channel Plan [1%] benefits amounted approximately to $2,941,000. A more detailed explanation of the methods and assumptions used to derive recreation benefits is contained within appendix 1 of the economic evaluation.

ADVANCE REPLACEMENT OF BRIDGES

The upper Guadalupe River Project will result in the replacement of existing bridge facilities. Some of these bridges will soon need to be replaced in the absence of the project due to obsolescence, age, changing needs, etc. The replacement of these bridges will extend the life of the bridge service currently being provided. Since the total cost of the new bridge is included in the first cost of the project, a credit for this extension of bridge life is needed on the benefit side. The 100-year plan involves the replacement of 5 bridges: the Willow Street Bridge, the

Alma Avenue Bridge, the Willow Glen Way Bridge, the Hillsdale Road Bridge, and the Curtner Avenue Bridge. The Valley View plan involves the replacement of 4 bridges: the Willow Street Bridge, the Alma Avenue Bridge, the Willow Glen Way Bridge, and the Hillsdale Road Bridge. The Willow Glen Plan involves the replacement of 2 bridges: the Willow Street Bridge, and the Alma Avenue Bridge

Even though the Hillsdale Bridge has no "useful remaining life" its inclusion in the Advance Bridge Replacement benefit category is consistent with the guidelines of IWR-88-R-2. The following hypothetical sample calculation is included in order to better illustrate the concept of bridge life extension and its application to the Hillsdale Bridge. The following example assumes the Hillsdale Avenue Bridge has one year of useful remaining life instead of the 0 years of remaining life that it currently has. Furthermore, because the cost of the new bridge will be included into the first cost of the project the benefits associated with the bridge services being rendered must also be included in the benefits.

Valley View Plan:
Hypothetical Benefit Derivation for Hillsdale Ave. Bridge - Assuming 1 year of remaining useful life as opposed to 0 Yrs..

1. Cost of new bridge		$1,336,603.00
2. Expected Life of the new bridge (in yrs.)	50	
3. Remaining useful life of current bridge (in yrs.)	1	
4. Extension of bridge life (1st through 50th year)		49.00
5. Discount Rate (FY 98)		0.07125
6. Capital recovery factor (for 50 years)		0.07361
7. Annual cost of new bridge	($1,336,603 x 0.07361) =	$98,387.00
8. Present worth of annuity factor for 49 yrs.		13.55362
9. Benefits in year 1, credited to bridge life extension		
	($98,387) x (13.55362)=	$1,333,505.00
10. Single payment present worth for 1 yrs.		0.93349
11.Present worth in year 1 of bridge extension		
	($1,335,505) x (0. 93349)=	$1,244,814.00
12.Average annual credit (benefit)	($1,244,814) x (.07361)=	$91,631.00

As can be seen from the sample calculation it is a simple process to use the same methodology to assess the benefit credit that would accrue to a bridge with 0 years of useful remaining life. In essence, we are extending the life of the Hillsdale Bridge 50 years into the future instead of the 49 in the sample calculation.

The costs of the new bridge include the estimated costs of contractor's overhead, contractor's profit, contractor's bond and contingencies.

This procedure was used to assess the benefit credit for each of the bridges to be replaced in each one of the alternatives. The benefits for the Valley View and Bypass Channel Plans are shown in Table 15.

Table 15
Advance Replacement of Bridges
Project Benefits

Bridge	Est. Remaining Years	Est. Max. Cap./lbs.	Bypass Channel Plan	Valley View Plan	Willow Glen Plan
Willow St.	5	8,000	$73,180	$146,250	$125,290
Alma St.	25	30,000	$27,680	$26,750	$24,030
Willow Glen Way	5	8,000	$120,960	$62,570	$0
Curtner Ave.	25	30,000	$29,460	$0	$0
Hillsdale Ave.	0	8,000	$292,680	$98,400	$0
Total			$543,960	$333,970	$149,320

Source: Economics Branch, San Francisco District
Price level = 1995

EMERGENCY COSTS TO FEMA

Emergency costs include losses over and above physical flood damages, which result from the disruption of normal activities. Emergency costs include the costs of evacuation and reoccupation of the floodplain, flood fighting, and disaster relief; increased costs of normal operations during the flood; and increased cost of police, fire, medical activity and military patrol. By providing flood protection, current costs for emergency aid would be saved. While many agencies and individuals are involved, for the purposes of this study, the benefits have been developed based on recent experiences of the Federal Emergency Management Agency (FEMA).

As a result of the 1995 winter storms in California, FEMA made direct payments of $81,000,000 to 29,200 families within the 42 counties that were declared flood disaster areas. These payments covered the cost of temporary housing, emergency repairs, "living kits", and crisis counseling. All of these expenditures are of the emergency-type and are separate and distinct from the flood damages, estimated elsewhere. In addition, there were administrative costs associated with the Disaster Assistance Program representing approximately 25 percent of the direct payment costs, or $20,000,000. Thus, the average payment per family is approximately $3,400, which is FEMA's total cost divided by the number of families receiving payments. It was assumed for the purposes of this analysis that families living in structures with the first floor below flood levels would experience emergency aid-type damages.

The NED benefits in savings for this program are presented in Tables 16a through 16c. As can be seen from this table, there are an estimated 712 structures which will be damaged above the first floor in a 20-year event; 1629 in a 50-year event, etc. The total emergency costs, which were calculated by multiplying the number of structures by the average payment per household ($3,400), vary from $2 million for a 20-year event to over $14 million for a 500-year event. The average annual savings in emergency costs are $312,000 for the Bypass Channel Plan [1%], $279,000 for the Valley View Plan [2%], and $269,000 for the Willow Glen [5%] Plan.

Table 16a
Number of Structures Experiencing First Floor Flooding

	Without Project	**With Project**		
Flood Event		**5%**	**2%**	**1%**
500-yr	**4,169**	**1,500**	**1,500**	**319**
100-yr	**2,527**	**691**	**434**	**228**
50-yr	**1,629**	**285**	**256**	**227**
20-yr	**712**	**0**	**0**	**0**

Source: Economics Branch, San Francisco District

Table 16b
Emergency Flood Costs

Flood Event	Prob.	Without Project	Project 5%	Project 2%	Project 1%
500-yr	0.002	$14,174,600	$5,100,000	$5,100,000	$1,084,600
100-yr	0.01	$8,591,800	$2,349,400	$1,475,600	$775,200
50-yr	0.02	$5,538,600	$969,000	$870,400	$771,800
20-yr	0.05	$2,420,800	$0	$0	$0
Ave. Ann. Costs		$341,628	$72,144	$62,308	$29,137

Price level = 1995

Table 16c
Average Annual Emergency Flood Benefits by Project

Project	5%	2%	1%
	$269,000	$279,000	$312,000

Price level = 1995

BENEFITS ACCRUED DURING CONSTRUCTION

According to ER 1105-2-100 benefits that accrue during project construction should be included in the NED calculations and the determination of net benefits. In this instance, significant benefits will begin to accrue to the Valley View or the Bypass Channel Plan during the construction period. These benefits are detailed in the following section.

Of the three alternatives presented in this analysis (the Willow Glen Plan, the Valley View Plan and the Bypass Channel Plan) two will begin to accrue significant project benefits during the construction period. Both the Valley View Plan and the Bypass Channel plan will begin to accrue project benefits approximately half way through the 3 year construction period. At this time these two alternatives will in fact provide protection which more than exceeds the protection provided by the Willow Glen Plan. The Willow Glen Plan provides for flood control works in reaches 7 and 8 and this level of protection results in approximately $15,134,000 in average annual benefits.

The Benefits Accrued During Construction for the Valley View Plan and the Bypass Channel plan are derived from the Willow Glen Plan. The construction schedule for these two alternatives calls for the construction of flood control works in reaches 7 and 8 which exceeds the protection parameters associated with the Willow Glen Plan. Furthermore, the construction schedule calls for completion of these works approximately half way through the three year construction period associated with the Valley View Plan and the Bypass Channel plan. This means that all of the annual benefits associated with the Willow Glen Plan would begin to accrue to the project once construction was completed in reaches 7 and 8.

To calculate the Benefits Accrued During Construction for the two alternatives in question a time period of 1.5 years was used. This period recognizes the fact that the project benefits begin to accrue even though construction has not been completed. In fact the protection provided by the Willow Glen plan will come on line 1.5 years before project completion. Using the Willow Glen benefits for 1.5 years ($15,134,000 X 1.5) and the capital recovery factor associated with the 50 year project life (0.07361), average annual benefits for the Benefits Accrued During Construction category were calculated. For this presentation it was assumed that the Benefits Accrued During Construction was the same for both the Valley View plan and the Bypass Channel plan. This assumption is considered conservative as the construction in reaches 7 and 8 associated with these two alternatives provides more protection than the Willow Glen Plan. The benefits were $1,671,021

BENEFIT SUMMARY

The flood control benefits summary is presented in Table 17a. Table 17b presents the benefit summary at the 1997 price levels. Under the future hydrologic flow regime, these benefits amount to $26,595,000 for the Bypass Channel Plan, $19,984,000 for the Valley View Plan and $15,134,000 for the Willow Glen Plan.

Table 17a
Summary of Flood Control Benefits (future)
1995 Price Levels, 7-1/8% Federal Discount Rate, 50-Year Project Life

Benefit Category	Willow Glen[5%]	Valley View[2%]	Bypass Channel[1%]
Flood Damage Reduction	$13,678,000	$16,405,000	$19,307,000
Federal Flood/Local Insurance Costs	$32,000	$57,000	$183,000
Current Maintenance	$120,000	$120,000	$200,000
Recreation Benefits	$0	$0	$2,941,000
Advance Replacement of Bridges*	$149,000	$334,000	$544,000
Emergency Flood Costs	$269,000	$279,000	$312,000
Total Flood Control Benefits	$14,248,000	$17,195,000	$23,487,000

*The Traffic Disruption Reduction benefits and the Benefits Accrued During Construction categories are not included in this table as they were completed in 97.

Table 17b
Summary of Flood Control Benefits (future)
1997 Price Levels, 7-1/8% Federal Discount Rate, 50-Year Project Life

Benefit Category	Willow Glen[5%]	Valley View[2%]	Bypass Channel[1%]
Flood Damage Reduction	$14,460,000	$17,343,000	$20,411,000
Federal Flood/Local Insurance Costs	$36,000	$65,000	$208,000
Traffic disruption Reduction Benefit	$74,000	$136,000	$179,000
Current Maintenance	$126,000	$126,000	$210,000
Recreation Benefits	$0	$0	$3,018,000
Advance Replacement of Bridges*	$156,000	$350,000	$570,000
Emergency Flood Costs	$282,000	$293,000	$328,000
Benefits Accrued During Construction	$0	$1,671,000	$1,671,000
Total Flood Control Benefits	$15,134,000	$19,984,000	$26,595,000

*The Traffic Disruption Reduction benefits and the Benefits Accrued During Construction categories are included in this summary table as they were completed subsequent to the 1995 benefit summary.

ECONOMIC COSTS

Traffic Re-routing and Delay Costs:

Each of the final alternatives includes bridge relocations and modifications. Bridge modifications may be accomplished without disruption to traffic. However, bridge replacements will result in temporary traffic detours. Each alternative has been designed to minimize disturbance of major traffic arteries and bridges. However, these delays would be considered an economic loss due to the cost of rerouting traffic, a loss of time to commuters and various other costs.

To reduce traffic disruption during construction, adjacent bridges will not be replaced simultaneously. Costs associated with traffic detours are summarized in Table 18.

TABLE 18

Traffic Re-Routing and Delay Costs

	Bridge to be Replaced	Alternate Bridge	Extra Distance Traveled (miles) 1/	Mileage ($/mile) 2/	Vehicles per Day 3/	Time Bridge Out (days)	Mileage Cost	Avg Delay (Min)	Avg Value of Time ($/Min)	Delay Cost	Total Cost
Willow	Willow Str.	W. Va. St.	0.55	0.165	7,401	90	$109,905	0.9	0.33	$ 207,249	$317,154
Glen	W. Alma Ave.	Willow Str.	1	0.3	6,108	90	$164,916	1.7	0.33	$ 310,984	$475,900
Plan							$274,821			$518,234	$793,054

	Bridge to be Replaced	Alternate Bridge	Extra Distance Traveled (miles)	Mileage ($/mile)	Vehicles per Day	Time Bridge Out (days)	Mileage Cost	Avg Delay (Min)	Avg Value of Time ($/Min)	Delay Cost	Total Cost
Valley	Willow Str.	W. Va. St.	0.55	0.165	7,401	90	$109,905	0.9	0.33	$ 207,249	$317,154
View	W. Alma Ave.	Willow Str.	1	0.3	6,108	90	$164,916	1.7	0.33	$ 310,984	$475,900
Plan	Wil. Glen Way	W. Alma Ave.	1.25	0.375	5,740	90	$193,725	2.1	0.33	$ 365,310	$559,035
	Hillsdale Ave.	Branham Ln.	1.7	0.51	9,520	90	$436,968	2.9	0.33	$ 823,997	$1,260,965
							$905,514			$1,707,540	$2,613,054

	Bridge to be Replaced	Alternate Bridge	Extra Distance Traveled (miles)	Mileage ($/mile)	Vehicles per Day	Time Bridge Out (days)	Mileage Cost	Avg Delay (Min)	Avg Value of Time ($/Min)	Delay Cost	Total Cost
Bypass	Willow Str.	W. Va. St.	0.55	0.165	7,401	90	$109,905	0.9	0.33	$ 207,249	$317,154
Channel	W. Alma Ave.	Willow Str.	1	0.3	6,108	90	$164,916	1.7	0.33	$ 310,984	$475,900
Plan	Wil. Glen Way	W. Alma Ave.	1.25	0.375	5,740	90	$193,725	2.1	0.33	$ 365,310	$559,035
	Hillsdale Ave.	Branham Ln.	1.7	0.51	9,520	90	$436,968	2.9	0.33	$ 823,997	$1,260,965
	Curtner Ave.	Malone Ave.	0.85	0.255	1,300	90	$29,835	1.5	0.33	$ 56,260	$86,095
							$935,349			$1,763,801	$2,699,150

1/ Distance is based on route traveled from point of westbound detour at bridge under construction, to alternate bridge, and back to point of eastbound detour at bridge under construction.

2/ Mileage rate = $0.30/mile.

3/ Based on 24-hour weekday volume traffic counts performed in April and December 1990.

Appendix 1

RECREATION BENEFITS

Various methods have been developed and tested over the years to assign monetary values to recreation use. The majority of these methods value recreational benefits by relating combinations of real and abstract factors to deduce a users willingness to pay. ER 1105-2-100 sets forth the following three basic methods to determine recreation benefits:

Travel Cost Methods (TCM). The basic premise of the TCM is that per capita use of a recreation site will decrease as out-of-pocket and time related costs of traveling to the site increase, other variables remaining constant. Using the out-of-pocket and time related costs that vary with trip distance, a demand curve can be derived that uses these costs (out-of-pocket expenses and trip distance related costs) as a proxy for price. The demand curve can then be used to derive consumer surplus, which in turn is used to derive the benefit related to the proposed recreation facility. Because the TCM's principle mechanism of deriving benefits is based on the variable costs of travel and the value of time, it is an inappropriate method of evaluation where there is very little variance in travel time required to access the site. As the proposed Guadalupe River Park will be in an urban setting, where most of the visitors will be local residents, workers and shoppers, there will be very little travel required to access the proposed recreation facilities; consequently, the TCM would be inappropriate for the evaluation of the recreation benefits associated with the proposed site.

Contingent Value Methods (CVM). Also referred to as the Survey Method in the past, this method relies on questionnaires to attempt to establish willingness to pay. Use of this method may be successful for specialized recreation activities such as hunting, white water boating and camping that are more market oriented; however, as the proposed facilities are more oriented to a general recreational experience, rather than a specialized one, this method was also deemed inappropriate.

Unit Day Value (UDV). This method relies on assigning point values to the various criteria that contribute to the quality and convenience of the recreation activity. This point total is then converted to a dollar value, which is used to determine benefits. Unit Day Value was deemed to be the most appropriate mode of evaluation for the proposed recreation site

SELECTION OF SUITABLE METHOD

The Unit Day Value Method (UDV) was used to establish recreation values for the Upper Guadalupe Flood Protection Study. For the reasons described in the above section the TCM and the CVM were deemed to be inappropriate methods of recreation benefit evaluation. In addition the selection of the UDV method conforms to the guidelines established in ER 1105-2-100 (Pg.6-84 and 6-85).

In order to establish the benefits associated with the proposed recreation facilities, using the UDV method, three basic components of analysis needed to be determined. First, a point system (Unit Day Values) was used to evaluate the total recreation experience. Second the point system is converted into dollars and third the estimated annual use was determined.

Determination of Unit Day Values.

Table 6-29 (Attachment A) in ER 1105-2-100 (Pg.6-113) outlines the categories, or Criteria, to be used in assigning points for general recreation activities. This point total is ultimately used to determine the dollar value for the recreation benefits. There are five criteria used in the evaluation, and each criterion is assigned a total number of possible points. The recreation resource is evaluated individually with respect to each one of the various criteria; each one of these categories is assigned a value from a range of appropriate values stipulated by the Principle and Guidelines document. Table 1 illustrates the assignation of points to the various categories.

Table 1
General Recreation Unit Day Values

Criteria	Possible Points	Point Range	Points assigned
Recreation Experience	**30**	**5-10**	**10**
Availability of Opportunity	**18**	**4-6**	**5**
Carrying Capacity	**14**	**9-11**	**11**
Accessibility	**18**	**15-18**	**18**
Environmental Quality	**20**	**17-10**	**10**
Total Point Allocation			**54**

RATIONALE FOR POINT VALUE ASSIGNMENT

(1) Recreation Experience (30 Points Maximum). The proposed park would provide the opportunity for several general outdoor activities, activities that would have not been available in one location. These general activities include walking, jogging, and picnicking, but not specialized activities uncommon to the region such as big game hunting, salmon fishing, and wilderness pack trips. A mid-range value of 10 was assigned for this category.

(2) Availability of Opportunity (18 Points Maximum). This criterion from a practical standpoint would apply predominantly to the local users. Several other locations that offer the same type of General Recreation are located within one hour travel time, however transportation may be a factor limiting use of other parks by the locals. A point value of 5 was assigned.

(3) Carrying Capacity (14 Points Maximum) Visitation or use estimates were made based on optimum carrying capacities being achieved considering the facilities and resources of the pathways. A score of 11 was assigned, indicating optimum facilities to conduct activity to site potential.

(4) Accessibility (18 Points Maximum) Accessibility to the park is ideal for all of the types of use and user expected. High water may limit activity in the site during the winter and early spring and this is reflected in the season use pattern. All the paths are accessible and provide links to major roads, therefore the maximum point value of 18 was assigned.

(5) Environmental Quality (20 Points Maximum) Environmental quality of the site including both the preserved and restored riparian areas as well as the built environment should qualify as above average. This is particularly significant in light of the highly urbanized surroundings. "Above average" esthetic quality lies in the middle of the rating system (low, average, above average, high, and outstanding) so a point value of 10 was assigned.

CONVERSION OF UNIT DAY VALUES TO DOLLARS

In order to establish the benefits for the proposed recreation area the Unit Day Value must be converted into dollars. Each Fiscal Year an Economic Guidance Memorandum is circulated to establish the current year's conversion rate. Economic Guidance Memorandum 95-3, Unit Day Values for Recreation, Fiscal Year 95 was used to establish the Unit Day Value of $5.40 for the proposed recreation area.

ESTIMATED ANNUAL USE

Previous Corps studies served as a guideline for developing the use rate for the upper Guadalupe River flood control project. The Design Capacity approach used in this recreational benefit analysis is modeled after a previous corps study on the Guadalupe River (Guadalupe, California. General Design Memorandum, December 1991). As part of the General Design Memorandum (GDM) an analysis of recreational use was conducted and a carrying capacity model was used as the basis for determining recreational benefits. After reviewing ER 1105-2-100, a carrying capacity model similar to the one used in the previous GDM was used to estimate the annual use rate of this recreation area.

A carrying capacity model evaluates a series of activities and derives a design capacity for each activity. The design capacity of an activity is the number of individuals who can participate in that activity, and receive the optimum recreation experience, at any one time. Because the design capacity is defined as the number of individuals who can derive maximum satisfaction at any given time it is assumed that the recreational behavior of visitors is indicative of their satisfaction. Therefore, the design capacities user load is based on the apparent preference of individuals to enjoy recreation experiences during peak summer months.

In order to estimate the annual use rate of the area, a design capacity (based on June through August) estimate was determined for each activity and the total of these individual design capacities gives the overall design capacity. After the total design capacity of 724 was derived, a turnover rate of three was applied to the design capacity to achieve the total Daily Design Load of 2,172. This methodology was used in the aforementioned Corps study and was deemed to be a reasonable approach towards deriving the annual use patterns of the proposed recreation system. These design capacities are given in Table 2.

Table 2
Design Capacity Analysis

Activity	Carrying Capacity	Units	Turnover Rate	Design Load
Walking	**50 Per Mile**	**5.5**	**3**	**825**
Jogging	**25 Per Mile**	**5.5**	**3**	**414**
Biking	**2 Per Mile**	**5.5**	**3**	**33**
Picnicking	**10 Per Mile**	**20**	**3**	**600**
Outside Sitting	**100**	**N/A**	**3**	**300**
Total Design Capacity	**2,172**			

In order to convert the daily design load into an annual use estimate, seasonal adjustments have to be made. The peak summer months are estimated to account for 35 percent of total annual park visitation. Therefore the total use is estimated by assuming that 35 percent of the annual visitors is equal to the daily design load of 2,172 multiplied by the 91 days of summer (June through August), which equals 203,000. This means that 198,000 is 35 percent of the total yearly visitation of 564,720. Estimated Annual Use is reported in Tables 3.

Table 3
Estimated Annual use

Daily Design Load	Season	Percent of Annual Use	Amount of Annual Use
	Mar thru May	25	141,180
2,172 x (91 days)	June thru Aug	35	197,652
	Sept thru Nov	25	141,180
	Dec thru Feb	15	84,708
		100	564,720
Existing Use			(20,000)
Total Estimated Annual Use			544,720

Given the proximity of the area to densely populated residential areas, it seems reasonable to anticipate that the capacity will be realized from the outset. Use surveys conducted recently at adjacent trails and parks support this conclusion. The recreational use at Los Alamitos/Calero Creek trail was recently estimated at 1200 users per day on a weekend and 900 a week day. This trail is further away from the population centers and is somewhat isolated. The location of the Guadalupe River recreation system is clearly superior. Another example of regional use rates is the Los Gatos Creek Trail. This trail is 10 miles away and the estimated use there for a 6 hour period yields a daily use rate of 4,500.

Moreover, this projects proximity to the new light rail system with an adjacent station is likely to draw more users. In addition, the San Jose master plan states that 2 % of the people who live within 5 miles of their job site commute to work via bicycle. As this 5 mile system connects residential areas in San Jose to Downtown San Jose, the use estimates presented here seem conservative. As the Guadalupe river Segment is optimally located and over 5 miles in length the use rate of 2,176 seems reasonable.

Benefit Calculation

The estimated Annual Use Benefit is calculated by multiplying the increase in estimated use due to project implementation by the determined Unit Day Value. The UDV, as stated earlier, was $5.40 and the net estimated use, as shown in Table 10, was 544,720. Multiplying these two factors gives an estimated annual benefit of $2,941,500.

Appendix 2

TRAVEL COST REDUCTION BENEFITS

During major flood events the Interstate 280 State Route 87 junction is flooded. Due to the configuration of the junction Interstate 280 is routed over the junction and is not affected by the flooding. However, State Route 87 is closed during flooding. This closure of State route 87 necessitates a detour around the flooded area and induces traffic delays. In addition to the delays experienced by motorists on State Route 87, the light rail system in the area will also be affected.

The benefits presented here consist of a time delay reduction benefit, a mileage traveled reduction benefit and a reduction in cleanup costs benefit.

By preventing flooding to the shortest route, a project creates a savings in travel time and, therefore, produces a benefit to the project. The following sections detail the assumptions and the data used to derive the average annual benefits attributable to project implementation. The methodology used in this analysis to derive the economic losses associated with traffic delays parallels the methodology presented in an IWR report (IWR Report 91-R-12, October 1991) entitled

VALUE OF TIME SAVED FOR USE IN CORPS PLANNING STUDIES
A REVIEW OF THE LITERATURE AND RECOMENDATIONS

Traffic Volume: The "1995 Traffic Volumes on California State Highways", published by the California State Transportation Department (CalTrans), was used to derive the traffic counts used in the analysis. This publication gives the volume of traffic on California highways at various points. In this particular instance the area of flooding, the Interstate 280 State Highway 87 junction, was directly listed in the publication. Table 1 lists the relevant volumes for State route 87 at this location.

Table 1

Average daily traffic volume

Peak hour	Peak month	Annual
6,000	67,000	60,000

To segregate the total volume of cars into a Peak hour load and a Non-Peak hour load the City of San Jose Transportation Department (the City) was contacted. The City stated that Peak morning traffic was between 6:30am and 9:00am, and that Peak afternoon traffic occurred between 3:00pm and 6:00pm. This means that there are 5.5 Peak hours of traffic and 18.5 Non-Peak hours of traffic. Table 2 presents the distribution of Peak and Non-Peak hours as well as the associated volume.

Table 2
Distribution of peak and non-peak hours in an Average day

	Number of hours	Volume per day
Peak Hours	5.5	33,000
Non Peak Hours	18.5	27,000

This distribution of vehicle volume was derived by multiplying the number of Peak hours by the Caltrans Peak hour load. This gave a daily load for the Peak hours of traffic. The Non-Peak hour's load was derived by subtracting the Peak hour's load from the daily Annual load.

In addition to the volume listed above it is expected that during a flood event the light rail service that runs through the junction will be suspended. It is assumed that this suspension will cause the affected ridership to use private vehicles for transportation. This increase in vehicular traffic was added to the State Highway 87 volume to create a total affected vehicle population.

The ridership data came from the Santa Clara Valley Transportation Authority (the Authority). Average daily ridership is approximately 20,000 per day. The Authority estimates that 65 percent of the ridership would be affected. It was therefore assumed that 65 percent of the affected ridership would use private transportation and that furthermore they would follow the detour pattern of the affected vehicles from State Route 87. In essence the affected ridership would add their volume to the volume of State Route 87. Table 3 shows the distribution of displaced riders for Peak and Non-Peak hours.

Table 3
Light Rail distribution of peak and non-peak Hours in an average day

	Number of hours	Volume per day
Peak Hours	5.5	7,150
Non-Peak Hours	18.5	5,850

The total affected vehicle population is the combination of vehicles displaced from State route 87 and the displaced ridership from the light rail system. Table 5 shows the total distribution used for the calculation of delay benefits. This distribution is consistent throughout the analysis. In other words the affected traffic volumes do not vary with the severity of the event. This is because once the route is severed and cars are detoured the alternative routes are expected to be clear for re-routing. This means that the volume of affected vehicles will not be greater for larger flood events. It is expected that a 20,50 or 100 year event will all be equally effective in closing State Route 87 and the Light Rail system. This closure will precipitate the rerouting of the total volume as shown in Table 4.

Table 4
Total distribution of peak and non-peak Hours in an average day

	Number of hours	Volume per day
Peak Hours	5.5	40,150
Non-Peak Hours	18.5	32,850

Length of Delay: Because no published data was available on the length of delay the City was contacted for an estimate of delays. The project manager for State Route 87 stated that the minimum delay experienced by vehicles would be no less than 30 minutes. Therefore the estimated length of delay used in the calculation of delay costs was 30 minutes per affected vehicle.

Length of State Route 87 closure: The length of closure was estimated by the Hydraulics & Coastal Section, San Francisco district. Using historical data the closure length was estimated for the 20, 50 and 100 year events. These closure lengths are listed below in Table 5.

Table 5

Estimate Traffic Delays

	Delay for a 20 Year Event	Delay for a 50 Year Event	Delay for a 100 Year Event
Closure In Days	2.5	3.5	5.5

Value of delay per minute: The value of the delay per minute figure was derived through a process which took into account the median family income of the area, the trip purpose of the affected population and the length of delay. The methodology used to calculate the value of the delay per minute is adapted from the IWR report mentioned previously. Essentially this method of valuation uses the median family income of the area as a proxy for the value of the affected drivers' income. Furthermore, this income is converted into a per minute delay cost by attributing different per minute values to different trip purposes. The following sections describe the data used to derive Median Family Income and the delay cost by trip purpose.

The Association of Bay Area Governments (ABAG) provided the base data for the determination of the Median family income. ABAG has conducted extensive studies of the San Francisco Bay Area Region and has published projected Median Family Income for a variety of years. ABAG's data was used to derive the 1997 Median family income. The 1997 Median family Income was derived by interpolating between ABAG's 1995 Median Family Income and the their year 2000 Median Family Income. The data and the 1997 Median Family Income are presented below in Table 6 and 7.

Table 6

Interpolated income from ABAG projections

Year	Estimate
1995 $	66,900
1996 $	68,560
1997 $	**70,020**
1998 $	71,580
1999 $	73,140
2000 $	74,700

Table 7

Calculation of Median Family Income Increments

Median Family Income (1997)	$70,020
Number of hours worked per year Assuming 8 hour/day *260 days/year	$ 2,080
Median Hourly Income	$ 33.66
Median Income per Minute	$ 0.56

IWR report 91-R-12 describes the derivation of the value of time lost and the derivation of the trip purpose categories. The value of time lost is a function of the length of the delay and the trip purpose. For this study Trip purpose was broken down into 4 categories. These Categories were:

1. Work Trips.
2. Social/Recreation Trips.
3. Other Trips
4. Vacation Trips

Each of these trip purposes is assigned a weight in terms of what percent of hourly income each type of delay is worth. Furthermore that weight is adjusted by the length of delay. As the delay increases the value of the time lost increases. IWR 91-R-12 specifies three trip length delay categories ranging from 0-5 minutes lost to over 15 minutes lost. As the delay in this study was estimated by the City to be 30 minutes the last category was used. Table 8 presents the percent of hourly income, which was used in this study.

Table 8
Value of Time by Trip Purpose

Category	**Income Adjustment Factor**
Work	53.8%
Personal business	60.0%
Social/Rec	64.5%
Vacation	75.1%

The percentages in Table 8 were applied to the per minute median income to achieve a total delay cost per minute for each of the respective trip categories. The total delay cost per minute is presented below in Table 9.

Table 9

Value of Time by Trip Purpose as Moderated by Length of Delay

Category	Income Adjustment Factor	Median Income per Minute	Adjustment Value of
Work	53.8%	$ 0.56	$ 0.30
Personal business	60.0%	$ 0.56	$ 0.34
Social/Rec	64.5%	$ 0.56	$ 0.36
Vacation	75.1%	$ 0.56	$ 0.42

*Work trip is on a per person basis while all other trip purposes are on a per vehicle basis. However, due to the unavailability of this information each auto was considered to have only one passenger

Calculation of losses: The calculation of losses is accomplished by combining the various loss factors. Losses were calculated for the 20, 50 and 100 year events. The difference in the calculation behind each of these three events is the length of closure for State Route 87. Tables 10a through 10c present the losses incurred under each of these scenarios.

Table 10a
20 Year event

Peak Hours						
	One Day Autos	Minutes of x Delay	Number of x Day	Number of x Per Vehicle	Value of Delay x Per minute	Losses
Work	36,135	30	2.5	1	$ 0.30	$ 818,049
Personal Business	2,008	30	2.5	1	$ 0. 34	$ 50,685
Social/Rec	2,008	30	2.5	1	$ 0.36	$ 54,486
None Peak Hours						
	One Day Autos	Minutes of x Delay	Number of x Day	Number of x Per Vehicle	Value of Delay x Per minute	Losses
Work	3,285	30	2.5	1	$ 0.30	$ 74,368
Personal Business	8,213	30	2.5	1	$ 0.34	$ 207,346
Social/Rec	16,425	30	2.5	1	$ 0.36	$ 445,794
Vacation	4,928	30	2.5	1	$ 0.42	$ 155,717
Peak Hour Losses						$ 923,219
Non-Peak Hours Looses						$ 883,224
Total of Peak and Non-Peak hour losses						**$ 1,806,443**

Table 10b

50 Year event

Peak Hours						
	One Day Autos	Minutes of x Delay	Number of x Day	Number of x Per Vehicle	Value of Delay x Per minute	Losses
Work	36,135	30	2.5	1	$ 0.30	$ 1,145,268
Personal Business	2,008	30	2.5	1	$ 0.34	$ 70,958
Social/Rec	2,008	30	2.5	1	$ 0.36	$ 76,280
None Peak Hours						
	One Day Autos	Minutes of x Delay	Number of x Day	Number of x Per Vehicle	Value of Delay x Per minute	Losses
Work	3,285	30	2.5	1	$ 0.30	$ 104,115
Personal Business	8,213	30	2.5	1	$ 0.34	$ 290,284
Social/Rec	16,425	30	2.5	1	$ 0.36	$ 624,111
Vacation	4,928	30	2.5	1	$ 0.42	$ 218,003
Peak Hour Losses						$ 1,2 92,507
Non-Peak Hours Looses						$ 1,236,514
Total of Peak and Non-Peak hour losses						**$ 2,529,021**

Table 10c

100 Year event

Peak Hours						
	One Day Autos	Minutes of x Delay	Number of x Day	Number of x Per Vehicle	Value of Delay x Per minute	Losses
Work	36,135	30	5.5	1	$ 0.30	$ 1,799,707
Personal Business	2,008	30	5.5	1	$ 0.34	$ 111,506
Social/Rec	2,008	30	5.5	1	$ 0.36	$ 119,869
None Peak Hours						
	One Day Autos	Minutes of x Delay	Number of x Day	Number of x Per Vehicle	Value of Delay x Per minute	Losses
Work	3,285	30	5.5	1	$ 0.30	$ 163,610
Personal Business	8,213	30	.5.5	1	$ 0.34	$ 456,161
Social/Rec	16,425	30	5.5	1	$ 0.36	$ 980,746
Vacation	4,928	30	5.5	1	$ 0.42	$ 342,577
Peak Hour Losses						$ 2,031, 082
Non-Peak Hours Looses						$ 1,943,094
Total of Peak and Non-Peak hour losses						**$ 3,974,176**

As can be seen from the above tables the key difference in the loss calculations is the length of closure. As the length of closure increases so do the losses. Table 11 below shows a summary of the time delay losses incurred for each event

Table 11

Summary of Traffic Delay Losses per Event

	20 Year Event	50 Year Event	100 Year Event
Losses	$ 1,806,443	$ 2,529,021	$ 3,974,176

Reduction in extra mileage driven: When the Interstate 280 State Route 87 junction is flooded traffic will have to detour to avoid the closed area. This detour will necessitate the incurrence of mileage over and above what would have been incurred had the detour not been in place. The City was contacted and asked which alternative routes might be used by traffic detoured from the junction. The City estimated that 2 alternative routes would be used, the Almaden Route and the Highway 82 Route. The first of these routes, the Almaden Route was not estimated to add any appreciable extra mileage over the "Without Project" Route. However the highway 82 Route was estimated to add 2.3 miles to the route. Furthermore it was estimated that 50 percent of the detoured vehicles would follow each route. This resulted in an average of 1.2 extra miles driven by each auto. Table 12 presents the mileage costs for each of the events under consideration.

The cost was derived by multiplying the number of detoured miles, by the number of affected cars, by the federal mileage rate, by the number of detour days.

Table 12

Calculation for Detour Mileage

	Mileage	Number of Cars	Cost per mile	Number of days	Losses
20 year event	1.2	73,000	0.31	2.5	$ 65,061
50 year event	1.2	73,000	0.31	3.5	$ 91,086
100 year event	1.2	73,000	0.31	5.5	$143,135

Cleanup costs: The implementation of a project would eliminate the need for all of the costs associated with cleaning up and re-opening the junction. The city was contacted for the State Route 87 cleanup costs incurred in 1995. The 1995 flood event was deemed to be approximately a 20 year event and the associated cleanup costs were approximately $100,000. Because, cleanup costs were not available for the 50 and 100 Year events they were estimated, to be $150,000 and $200,000 respectively.

Summary of total losses: Table 13 below presents the overall losses incurred for a 20, 50 and 100 year event.

Table 13

Overall Losses per Event

Category	**20 Year**	50 Year	100 Year
Time Delays	$ 1,806,443	$2,529,021	$3,974,176
Mileage Costs	$ 65,061	$ 91,086	$ 143,135
Cleanup Costs	$ 100,000	$ 150,000	$ 200,000
Total Costs	**$1,971,505**	**$2,770,107**	**$4,317,310**

Average annual benefits: The average annual benefits were determined by using a damage frequency integration technique utilizing the zero damage point associated with the "Without Project" condition and the zero damage point of the "With Project" condition. The "Without Project" zero damage point was determined to be a 13.5-year (7.4 percent exceedance) event by the San Francisco Districts Hydraulics & Coastal section. The "With Project" zero damage point for each of the proposed projects is the level of protection they provide. Hence the Willow Glen plans zero damage point is the 5 percent probability exceedance event. Likewise, the Valley View and the By-pass channel plans have zero damage points at the 2 and 1 percent probability exceedance points.

To derive the AAE benefits, the "Without Project" AAE damages were derived utilizing the Quadrant Curve method of analysis. After graphing the "Without Project" damage curve it was integrated to derive the Average Annual Damages of the "Without Project" condition. Following this procedure the residual damage curves associated with each of the alternatives was plotted on the same graph. This process identified the increment of benefit attributable to each of the 3 alternatives. Table 14 presents the Average Annual Damages, which amount to $105,000, $43,000, and $0 for the 20-year, 50-year and 100-year projects, respectively. Table 15 presents the Average Annual Benefits associated with the reduction of travel disruption costs.

Table 14
Average Annual Traffic Losses

	With Project Residuals		
Without Project	20-Year Project	50-Year Project	100-Year Project
$179,000	$105,000	$43,000	$0

Table 15
Average Annual Traffic Disruption Reduction Benefits

	Willow Glen Plan	Valley View Plan	Bypass Channel Plan
Benefits	$74,000	$136,000	$179,000

UPPER GUADALUPE
FINANCIAL ASSESMENT
JANUARY 1998

The objective of this analysis is to conduct an initial financial assessment of the local sponsor for the Upper Guadalupe Flood Control Project. This initial assessment is intended to demonstrate that the cost sharing partner, the Santa Clara Valley Water District (the Water District), has successfully met its financial commitments in the past, has a variety of funding sources available to it, and has the capacity to ensure that the non-federal portion of the project funds will be available.

The first project cost estimate for the recommended Bypass Channel Plan, which is a Locally Preferred Plan (LPP), is approximately $132,298,000. This project provides protection against the 1 percent probability exceedance event. The first project cost estimate for the NED Plan is approximately $83,154,000. This project provides protection against the 2 percent probability exceedance event. Based on the increment between the NED and LPP costs, the Federal share of the recommended plan would be $54,050,000, while the non-Federal share would be $78,248,000.

Table 1 shows the projected schedule of Federal and Non-Federal expenditures required for project completion. The final details of the cost sharing mix and the schedule of expenditures will be specified by the Project Cooperation Agreement (PCA).

TABLE 1
Schedule of Estimated Federal and Non-Federal Expenditures
Flood Control @ 98.4% and Recreation @ 1.6%

Fiscal Year	Federal	Non-Fed Cash	Non-Fed Lands	Non-Fed Relocations
FY98	$600,000	$200,000		
FY99	$900,000	$300,000		
FY00	$1,125,000	$375,000		
FY01	$413,000	$0	$49,496,000	$3,671,000
FY02	$17,004,000	$4,397,000		$3,671,000
FY03	$17,004,000	$4,397,000		$3,671,000
FY04	$17,004,000	$4,397,000		$3,672,000
FY05				
Subtotal	$54,050,000	$14,066,000	$49,496,000	$14,685,000

Note: Table 1 costs are presented at the 1997 Price Level.

Total Sponsor Cost = $78,248,000
Total Federal Cost = $54,050,000
Total Project Cost = $132,298,000

Federal Project Cost Percentage = 40.9%
Non-Federal Project Cost Percentage = 59.1%

Table 1 Notes:

C:\Work\GUADLUPE\Financial assessment\FA1.xlsTable
1a!R1C1:R19C9

Fiscal Year 1998. Initiate PED.		**$ 800,000**
Fiscal Year 1999. Continue with PED, prepare General Design		**$ 1,200,000**
Fiscal Year 2000. Continue with PED, plans and		**$ 1,500,000**
Fiscal Year 2001. Sponsors purchase of project and mitigation	$49,496,000	
1st year of utility	$ 3,671,000	
Plus federal administration	$ 413,000	
subtotal		**$ 53,580,000**
Fiscal Year 2002. First Year Of Construction.	$21,401,000	
Plus 2nd year of utility	$ 3,671,000	
subtotal		**$ 25,072,000**
Fiscal Year 2003. Second year Of Construction	$21,401,000	
Plus 3rd year of utility	$ 3,671,000	
subtotal		**$ 25,072,000**
Fiscal Year 2004. Complete Construction	$21,401,000	
Plus 4th year of utility	$ 3,672,000	
subtotal		**$ 25,073,000**
Fiscal Year 2005. Closeout construction.		
Fiscal Year 2006 and thereafter Estimated future annual operation and maintenance		

PRIOR CORPS COOPERATION

The Water district has successfully cooperated with the Corps of Engineers on several previous occasions. The local sponsor (the Water District) has met all of its obligations with regard to cost sharing the present feasibility study. They have also fulfilled all of their obligations with regards to a prior Corps study and subsequent construction on the Guadalupe River. The Downtown project (which provides flood protection to downtown San Jose) is currently under construction and has been successfully cost shared at all appropriate milestones. In addition to these projects the Water district has fulfilled all of its financial obligations with regards to the Coyote Creek project. The successful participation of the local sponsor in these and other projects indicates the Water Districts good faith effort to meet its financial obligations in its dealings with the Corps of Engineers.

FINANCIAL INDICATORS

Several financial indicators were evaluated to give a broad indication of the financial health of the Water District. The Cash Ratio gives an indication of the Water districts immediate ability to repay debt. This ratio is defined as

Cash Ratio = Cash + Marketable Securities / Current Liabilities.

This ratio for the Water district is rated as strong with a ratio of 4.39. A ratio of over 1 is generally thought to be satisfactory. Therefore, the Water Districts rating of 4.39 is assessed to be strong.

The Debt to Equity ratio is also indicative of an entity's ability to repay debt. This ratio is defined as

Debt to Equity Ratio = Total Liabilities / Equity

This ratio for the Water district is also rated as strong at 0.28. A rating of under 0.5 is generally thought to indicate a strong ability to repay debt as it measures the overall ratio of debt to equity in the entity.

The last debt related ratio evaluated was the Debt Ratio. This ratio is defined as

Debt Ratio = Total Liabilities / Total Assets

This ratio for the Water district was also rated as strong. A rating of under 0.5 is usually deemed to indicate a strong balance sheet and a high probability of repayment. The water districts ratio was 0.22

The Financial Statements of the Water District have been included in the section titled

Attachment A
Financial Reports

FUNDING SOURCES

The Water Districts operation and programs are funded through a variety of sources. In any given year these sources can include the following revenue generating vehicles among miscellaneous other sources:

1. Property Taxes
2. Interest Income
3. Contributions and Reimbursements
4. Water Revenue
5. Benefit Assessments
6. Bonds and Certificates of Cooperation.

A portion of the county's property taxes are allocated to the SCVWD. These revenues generated from the collection of property taxes are distributed from the county to the Water District.

Interest income is derived from the investment of funds not needed for the day to day operations of the Water District. All of the investments have the stated goal of providing safety, liquidity and yield in that order. These investment vehicles include US Treasuries, Certificates of Deposit and other instruments consistent with the stated goals of the fund.

Another source of potential revenue comes from contributions and reimbursements from other governmental bodies and private industries.

Direct revenues from wholesale water distribution are another source of revenues. The Water District wholesales water to 13 municipalities and privately held water companies.

Benefit assessments are also a source of revenue. Voter approved benefit assessments are levied on all land parcels within the Water Districts jurisdiction. These benefit assessments raise revenue based on an estimate of the parcels runoff.

Bonds and Certificates of Participation offer the Water District another source of funds when required. The Water District issues Bonds and certificates of Participation to finance certain flood control and water utility projects. The Bonds are rated AA and are considered to be "investment grade". The Certificates of Participation are borrowing instruments approved by the voters of the Water District. They authorize the Water District to issue Certificates of Participation in order to finance water resource projects.

The exact combination of financial vehicles used to fund the local sponsor's portion of project costs will be determined at the time of the PCA signing.

CONCLUSION

The local sponsors have the capability to incur and sustain debt in excess of the non-federal share of the project cost. The actual funding mechanism or combination of funding mechanisms to be used by the local sponsor will be determined when the PCA is signed. The local sponsors have a strong financial position and sound financial statements. The aforementioned financial ratios indicate the strong nature of the Water district's financial statements and its ability to pay the non-federal portion of project costs. In aggregate the local sponsor has the financial wherewithal to provide the funds for the non-federal project cost. These funds may or may not derive from debt instruments.

ATTACHMENT A
FINANCIAL REPORTS

SANTA CLARA VALLEY WATER DISTRICT

Macias, Gini & Company LLP
Certified Public Accountants

Partners

Kenneth A. Macias
Ernest J. Gini

2855 Mitchell Drive
Suite 235
Walnut Creek, CA 94596

510•274•0190
510•256•7748 FAX

SANTA CLARA VALLEY WATER DISTRICT

Independent Auditor's Report, General-Purpose Financial Statements, and Supplementary Information

For the Year Ended June 30, 1996

OFFICE LOCATIONS

Sacramento • Los Angeles • Orange County • San Francisco Bay Area

SANTA CLARA VALLEY WATER DISTRICT

Combined Balance Sheet
All Fund Types and Account Groups
June 30, 1996
(Dollars in Thousands)

	Governmental Fund Types		
	General	Special Revenue	Capital Projects
ASSETS AND OTHER DEBITS:			
Assets:			
Cash and investments	$ 30,859	$ 106,527	$ 39,996
Restricted cash and investments	-	6,622	11,352
Receivables:			
Accounts	64	8	-
Interest	4,116	1	430
Taxes	3	24	-
Due from other governments	39	41,094	-
Condemnation deposits	-	43	20
Prepaid assets	33	-	-
Inventory	-	-	-
Property, plant and equipment, net	-	-	-
Contract water rights, net	-	-	-
Deferred charges	-	-	-
Due from others	-	-	-
Deferred compensation plan assets	-	-	-
Other debits:			
Amount available for debt service	-	-	-
Amount to be provided for repayment of long-term debt	-	-	-
Total assets and other debits	$ 35,114	$ 154,319	$ 51,798

See accompanying notes to the general-purpose financial statements.

Proprietary Fund Types		Fiduciary Fund Type	Account Groups		
Water Enterprise Fund	Internal Service	Agency	General Fixed Assets	General Long-Term Debt	Total (Memorandum Only)
$ 127,335	$ 16,326	$ 2,950	$ -	$ -	$ 323,993
7,661	-	-	-	-	25,635
8,834	-	365	-	-	9,271
10	-	-	-	-	4,557
114	-	-	-	-	141
126	-	-	-	-	41,259
-	-	-	-	-	63
-	25	-	-	-	58
200	24	-	-	-	224
289,273	4,748	-	429,223	-	723,244
16,953	-	-	-	-	16,953
182	-	-	-	-	182
-	-	78	-	-	78
-	-	17,950	-	-	17,950
-	-	-	-	11,352	11,352
-	-	-	-	129,083	129,083
$ 450,688	$ 21,123	$ 21,343	$ 429,223	$ 140,435	$ 1,304,043

(Continued)

SANTA CLARA VALLEY WATER DISTRICT
Combined Balance Sheet (Continued)
All Fund Types and Account Groups
June 30, 1996
(Dollars in Thousands)

	Governmental Fund Types		
	General	Special Revenue	Capital Projects
LIABILITIES, EQUITY AND OTHER CREDITS:			
Liabilities:			
Accounts payable	$ 1,739	$ 2,114	$ 2
Accrued liabilities	400	1,775	-
Deposits payable	-	6,051	-
Bond interest payable	-	-	-
Deferred revenue	39	26,572	-
Claims payable	-	-	-
Deferred compensation payable	-	-	-
Long-term debt:			
General obligation bonds	-	-	-
Revenue Bonds	-	-	-
Certificates of participation	-	-	-
Loans payable	-	-	-
Total liabilities	2,178	36,512	2
Equity and other credits:			
Investment in general fixed assets	-	-	-
Contributed capital	-	-	-
Retained earnings	-	-	-
Fund balances:			
Reserved for:			
Encumbrances	3,014	12,079	-
Noncurrent assets	-	5,575	-
Restricted purposes	-	3,600	20
Debt service	-	2	11,352
Unreserved, designated for:			
Operating contingencies	200	488	-
Working capital	975	13,951	-
Current and future construction	28,747	82,112	40,424
Total equity and other credits	32,936	117,807	51,796
Total liabilities, equity and other credits	$ 35,114	$ 154,319	$ 51,798

See accompanying notes to the general-purpose financial statements.

Proprietary Fund Types		Fiduciary Fund Types	Account Groups		
Water Enterprise Fund	Internal Service	Agency	General Fixed Assets	General Long-Term Debt	Total (Memorandum Only)
$ 5,407	$ 662	$ 115	$ -	$ -	$ 10,039
2,697	126	-	-	-	4,998
104	-	3,278	-	-	9,433
427	-	-	-	-	427
126	-	-	-	-	26,737
-	4,157	-	-	-	4,157
-	-	17,950	-	-	17,950
				-	-
17,835	-	-	-	1,625	19,460
50,614	-	-	-	-	50,614
-	-	-	-	138,810	138,810
158	-	-	-	-	158
77,368	4,945	21,343	-	140,435	282,783
-	-	-	429,223	-	429,223
4,378	-	-	-	-	4,378
368,942	16,178	-	-	-	385,120
-	-	-	-	-	15,093
-	-	-	-	-	5,575
-	-	-	-	-	3,620
-	-	-	-	-	11,354
-	-	-	-	-	688
-	-	-	-	-	14,926
-	-	-	-	-	151,283
373,320	16,178	-	429,223	-	1,021,260
$ 450,688	$ 21,123	$ 21,343	$ 429,223	$ 140,435	$ 1,304,043

SANTA CLARA VALLEY WATER DISTRICT

Combined Statement of Revenues, Expenditures and Changes in Fund Balances
All Governmental Fund Types
For the Year Ended June 30, 1996
(Dollars in Thousands)

	Governmental Fund Types			
	General	Special Revenue	Capital Projects	Total (Memorandum Only)
Revenues:				
Property taxes	$ 1,861	$ 13,537	$ -	$ 15,398
Benefit assessments	-	20,515	-	20,515
Intergovernmental	4	822	-	826
Use of money and property:				
Interest	1,844	6,459	3,971	12,274
Rental	40	738	-	778
Reimbursement of capital costs	-	10,973	-	10,973
Other	42	57	-	99
Total revenues	3,791	53,101	3,971	60,863
Expenditures:				
Current:				
Operations and maintenance	949	13,188	-	14,137
Operating projects	653	4,460	-	5,113
Capital improvement projects	2,149	43,635	-	45,784
Debt service:				
Principal repayment	198	2,756	-	2,954
Interest and fiscal charges	573	7,546	6	8,125
Total expenditures	4,522	71,585	6	76,113
Excess (deficiency) of revenues over (under) expenditures	(731)	(18,484)	3,965	(15,250)
Other financing sources (uses):				
Operating transfers in	3,346	1,139	-	4,485
Operating transfers out	(530)	(2,031)	(682)	(3,243)
Total other financing sources (uses)	2,816	(892)	(682)	1,242
Excess (deficiency) of revenues and other financing sources over (under) expenditures and other financing uses	2,085	(19,376)	3,283	(14,008)
Fund balances, beginning of year	30,851	137,183	48,513	216,547
Fund balances, end of year	$ 32,936	$ 117,807	$ 51,796	$ 202,539

See accompanying notes to the general-purpose financial statements.

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SANTA CLARA VALLEY WATER DISTRICT

Combined Statement of Revenues, Expenditures and Changes in Fund Balances
Budget and Actual
All Budgeted Governmental Fund Types
For the Year Ended June 30, 1996
(Dollars in Thousands)

	General Fund		
	Budget	Actual	Variance Favorable (Unfavorable)
Revenues:			
Property taxes	$ 1,950	$ 1,861	$ (89)
Benefit assessments	-	-	-
Intergovernmental services	-	4	4
Use of money and property:			
Interest	903	1,844	941
Rental	60	40	(20)
Reimbursement of capital costs	-	-	-
Other	-	42	42
Total revenues	2,913	3,791	878
Expenditures:			
Current:			
Operations and maintenance	31,953	30,931	1,022
Operating projects	1,234	661	573
Capital improvement projects	24,083	4,094	19,989
Debt service	770	771	(1)
Total expenditures	58,040	36,457	21,583
Other financing sources:			
Intra-district overhead reimbursement	36,336	30,262	(6,074)
Excess (deficiency) of revenues over (under) expenditures	$ (18,791)	(2,404)	$ 16,387
Fund balances, beginning of year		30,851	
Reconciliation of GAAP and budgetary basis:			
Operating transfers in		3,346	
Operating transfers out		(530)	
Less GAAP basis accruals for salary and other liabilities		(128)	
Add current year budgetary basis encumbrances		2,757	
Less prior year budgetary basis encumbrances		(956)	
Fund balances, end of year		$ 32,936	

See accompanying notes to the general-purpose financial statements.

Special Revenue Funds			Total (Memorandum Only)		
Budget	Actual	Variance Favorable (Unfavorable)	Budget	Actual	Variance Favorable (Unfavorable)
$ 14,096	$ 13,537	$ (559)	$ 16,046	$ 15,398	$ (648)
21,101	20,515	(586)	21,101	20,515	(586)
604	822	218	604	826	222
3,320	6,459	3,139	4,223	8,303	4,080
630	738	108	690	778	88
10,549	10,973	424	10,549	10,973	424
-	57	57	-	99	99
50,300	53,101	2,801	53,213	56,892	3,679
16,161	13,234	2,927	48,114	44,165	3,949
5,119	4,374	745	6,353	5,035	1,318
97,002	54,924	42,078	121,085	59,018	62,067
10,320	10,302	18	11,090	11,073	17
128,602	82,834	45,768	186,642	119,291	67,351
-	-	-	36,336	30,262	(6,074)
$ (78,302)	(29,733)	$ 48,569	$ (97,093)	(32,137)	$ 64,956
	137,183			168,034	
	1,139			4,485	
	(2,031)			(2,561)	
	(37)			(165)	
	11,567			14,324	
	(281)			(1,237)	
	$ 117,807			$ 150,743	

SANTA CLARA VALLEY WATER DISTRICT

Combined Statement of Revenues, Expenses and
Changes in Retained Earnings
All Proprietary Fund Types
For the Year Ended June 30, 1996
(Dollars in Thousands)

	Water Enterprise Fund	Internal Service	Total (Memorandum Only)
Operating revenues:			
Ground water charges	$ 29,302	$ -	$ 29,302
Treated water charges	33,450	-	33,450
Surface and reclaimed water charges	254	-	254
Vehicle service charges	-	2,481	2,481
Self-insurance service charges	-	3,172	3,172
Other	1,708	2	1,710
Total operating revenues	64,714	5,655	70,369
Operating expenses:			
Source of supply	31,839	-	31,839
Water treatment	12,344	-	12,344
Transmission and distribution:			
Raw water	4,628	-	4,628
Treated water	734	-	734
Administration and general	8,798	1,276	10,074
Equipment maintenance	1,232	2,258	3,490
Insurance	-	2,557	2,557
Depreciation and amortization	6,090	626	6,716
Total operating expenses	65,665	6,717	72,382
Operating (loss)	(951)	(1,062)	(2,013)
Nonoperating revenues:			
Property taxes	16,349	-	16,349
Interest	7,771	857	8,628
State Water Project reimbursements	2,185	-	2,185
Total nonoperating revenues	26,305	857	27,162
Nonoperating expenses:			
Interest	4,249	-	4,249
Fiscal agent fees	10	-	10
Total nonoperating expenses	4,259	-	4,259
Net income (loss) before operating transfers	21,095	(205)	20,890
Operating transfers in	-	1,411	1,411
Operating transfers (out)	(2,528)	(125)	(2,653)
Net income	18,567	1,081	19,648
Retained earnings, beginning of year	350,375	15,097	365,472
Retained earnings, end of year	$ 368,942	$ 16,178	$ 385,120

See accompanying notes to the general-purpose financial statements.

SANTA CLARA VALLEY WATER DISTRICT

Combined Statement of Cash Flows
All Proprietary Fund Types
For the Year Ended June 30, 1996
(Dollars in Thousands)

	Water Enterprise Fund	Internal Service	Total (Memorandum Only)
Cash flows from operating activities:			
Operating (loss)	$ (951)	$ (1,062)	$ (2,013)
Adjustments to reconcile operating (loss) to net cash provided by operating activities:			
Depreciation and amortization	6,090	626	6,716
Loss from sale of assets	22	22	44
Changes in operating assets and liabilities:			
Decrease (increase) in accounts receivable	(694)	13	(681)
Decrease in due from other governments	1,271	-	1,271
Decrease in prepaid purchased water	1,367	-	1,367
Decrease in inventory	230	15	245
Increase in accounts payable	2,823	180	3,003
Increase in accrued liabilities	106	5	111
(Decrease) in deposits payable	(2,305)	-	(2,305)
Increase in deferred revenue	13	-	13
Increase in claims liabilities	-	1,289	1,289
Net cash provided by operating activities	7,972	1,088	9,060
Cash flows from noncapital financing activities:			
Property taxes received	16,235	-	16,235
Operating transfers from (to) other funds	(2,528)	1,286	(1,242)
Net cash provided by noncapital financing activities	13,707	1,286	14,993
Cash flows from capital and related financing activities:			
Principal payments on bonds	(1,360)	-	(1,360)
Principal payments on revenue refunding bonds	(5,715)	-	(5,715)
Principal payments on loans payable	(84)	-	(84)
Interest and fiscal agent fees paid	(3,773)	-	(3,773)
State Water Project reimbursements	2,185	-	2,185
Acquisition of contract water rights	(1,618)	-	(1,618)
Purchase of fixed assets	(19,445)	(1,718)	(21,163)
Net cash used by capital and related financing activities	(29,810)	(1,718)	(31,528)
Cash flows from investing activities:			
Net change in restricted cash and investments	-	5	5
Interest received on cash and investments	10,192	1,139	11,331
Net cash provided by investing activities	10,192	1,144	11,336
Net increase in cash and cash equivalents	2,061	1,800	3,861
Cash and cash equivalents, beginning of year	132,935	14,526	147,461
Cash and cash equivalents, end of year	$ 134,996	$ 16,326	$ 151,322

See accompanying notes to the general-purpose financial statements.

ATTACHMENT B

LETTER OF INTENT
SANTA CLARA VALLEY WATER DISTRICT

Santa Clara Valley Water District

5750 ALMADEN EXPRESSWAY
SAN JOSE, CA 95118-3686
TELEPHONE (408) 265-2600
FACSIMILE (408) 266-0271

AN AFFIRMATIVE ACTION EMPLOYER

December 12, 1997

United States Army Corps of Engineers
Attention: Mr. Thomas R. Kendall
Acting Chief, Planning Engineering Division
333 Market Stree, Seventh Floor
San Francisco, CA 94105-2197

Dear Mr. Kendall:

The Santa Clara Valley Water District has been working with the Corps of Engineers for the past several years on the feasibility study of the Upper Guadalupe River Project. This very important project is located upstream of the Guadalupe River Project through downtown San Jose and it would provide increased flood protection to over 6,600 homes and businesses in the City of San Jose.

The District continues to be very interested in seeing this project implemented and it is our intention to actively sponsor this project. We realize that funding commitments of the District cannot be made, however, until the project limits, project cost, and cost sharing formula are determined. Once these matters are determined, the District would be able to establish its funding capability for this project and work on the necessary agreements with the Corps for cost sharing to construct, operate, and maintain it..

We look forward to timely resolution of the project cost issues so that progress can be made on this critical project.

Sincerely,

P. Kay Whitlock

P. Kay Whitlock
Assistant General Manager

recycled pape

COST ESTIMATES

The channel work costs include some excavation costs which may be eliminated during the preliminary engineering design (PED) phase. Current soils information indicates that bank stabilization measures would be required where cribwalls, gabions, and articulated concrete mats are employed. During construction in these areas, the bank must be excavated up to 60 feet from the existing bank. After geogrid stabilizing material is put in place, the area would be backfilled with compacted soil. The 60-foot cutback would impact streets and private yards in some areas. These costs are included in the cost estimate figures. However, no additional real estate easements or mitigation costs are included in the estimates. Instead, it is proposed that an in situ soil mixed wall be constructed as a retaining wall which would reduce the required cutback. This wall would be permanently buried in place and constructed using a drilling technique, rather than traditional excavation. During the PED phase, the need for this wall will be evaluated. Additional soil tests are planned to determine the extent of backfill required. Furthermore, adjustments to the alignment will be explored to determine whether the wall is required.

MCACES PRELIMINARY COST ESTIMATES

UPPER GUADALUPE RIVER

BASELINE DATE: OCT. 1997

PREPARED 20 AUG. 1997

MCACES
PRELIMINARY COST ESTIMATE

UPPER GUADALUPE RIVER
FLOOD PROTECTION PLAN PROJECT
SANTA CLARA COUNTY, CALIFORNIA

TO THE BEST OF MY KNOWLEDGE THE ESTIMATE WAS PREPARED IN ACCORDANCE WITH EC 1110-2-263 AND EC 1110-2-538.

ESTIMATED BY:
EDGARDO A. GONZALES
ESTIMATOR, EST. & SPEC. SECT.
DATE:

REVIEWED BY: Philip G. Parry
for CHRISTOPHER O. SHIELDS
CHIEF, SPECIFICATION AND COST
ENGINEERING SECTION
DATE: 8/25/97

K P Kuhn
REVIEWED BY: KENNETH P. KUHN
CHIEF, ENGINEERING BRANCH
DATE:

UPPER GUADALUPE RIVER PROJECT
BASIS OF COST

GENERAL

1. Project Description: The Upper Guadalupe River project is a Flood Control project designed to improve the river flow by widening or using a bypass channel. The project consists of alternatives from river widening to ~~using~~ a bypass channel, or a combination of both, while improving the erosion control of the river bank.

2. This estimate is based on an October 1997 price level. The original estimate has been escalated 2.8% from December 1996 to an October 1997 price level. Escalation factor is from "CWCCIS, EM 1110-2-1304, 9/97". Plant and equipment costs were from the UPB 1994 database for Region VII, and had been escalated to December 1996 cost. Labor costs were from the State of California wage rate determination sheets of August 1996.

3. The bridge removal and replacement cost were based on the previous study, escalated to December 1996. There is no bridge design furnished to do a detail cost estimates of the bridges. The cost of culverts are assumed from suppliers or from place of manufacture to the project site. The temporary relocation of culverts and railroad includes all the cost associated with construction.

4. Channel Lining Modifications include rock linings, gabions, and access ramps. The cost of waterline relocations is provided by the San Jose Water Co. Excavations and fills will be done using one equipment with 3 cubic yard bucket or blade. Structural excavations will take approximately 22 months with one equipment and may be done concurrent with other excavations with more than one equipment. Cost of excess excavated materials are based on loading onto trucks, hauled and disposed at Guadalupe Landfill or other landfill sites if available.

5. Real Estate costs were provided by the Sacramento District which include: Appraisal, Rights of Way/Temporary Permit, Land Acquisition cost, Removal/relocation of residential and commercial sites, and other Administration costs.

6. The estimate assumes that the Contractor will do approximately 90% of the job and sub contract 10%. The estimate assumes that the Contractor will be working 8 hours a day, 5 days a week. The project will be performed in one contract.

7. Mobilization and demobilization for plant and equipment is based on the preparation, transfer, set-up and removal of plant and equipment required. It is assumed that initial mobilization and demobilization of equipment can be performed in 3 days and 1 day thereafter between each site.

8. The midpoint date of construction is November 2003. The fully funded estimate has been escalated 19.41% from the October 1997 price level.

Profit Weighted Guidelines
From MCACES

1. Risk: If the work involves no risk or the degree of risk is very small, the weighting should be .03; as the degree of risk increases, the weighting should be increased up to a maximum of .12. Some of the other things to consider: the amount of work to be performed by subcontractors, the nature of work, where the work is to be done, and the amount of labor included in the costs.

2. Difficulty: If the work is most difficult and complex the weighting should be .12 and should be reduced proportionately to .03 on the simplest jobs. Some of the things to consider: the nature of the work, by whom and where the work is to be done, and what is the time schedule.

3. Size: All work not in excess of $100,000 shall be weighted at .12. Work estimated at between $100,000 and $5,000,000 shall be weighted proportionately from .12 to .05. Work from $5,000,000 to $10,000,000 shall be weighted at .04, and work in excess of $10,000,000 at .03.

4. Period: Jobs in excess of 24 months are to be weighted at .12 and jobs of lesser duration are to be proportionately weighted to a minimum of .03.

5. Investment: Contractor's investments are to be weighted from .03 to .12 on the basis of below average, to above average. Things to consider: amount of work to be subcontracted, mobilization payment item, and government furnished property.

6. Government assistance: To be weighted from .12 to .03 on the basis of average to above average. Things to consider: use of government-owned property, equipment and facilities, etc.

7. Subcontracting: Subcontracting value is based on the amount of work to be subcontracted. Where 80% or more of work subcontracted should be weighted .03, and increased proportionately to .12 where all work is performed by the prime contractor's own forces. See the following percentage table.

80% or more	0.030
70% to 80%	0.042
60% to 70%	0.055
50% to 60%	0.068
40% to 50%	0.080
30% to 40%	0.092
20% to 30%	0.105
10% to 20%	0.118
0% to 10%	0.120

Profit Weighted Calculations for Upper Guadalupe River Project

					Possible Range for this project
1.	Risk :	0.080 x 20 =	1.60%	----	0.070 - 0.100
2.	Diff :	0.080 x 15 =	1.20%	----	0.070 - 0.100
3.	Size :	0.030 x 15 =	0.45%	----	0.030 - 0.050
4.	Period :	0.110 x 15 =	1.65%	----	0.090 - 0.120
5.	Invest :	0.050 x 5 =	0.25%	----	0.040 - 0.070
6.	Assist :	0.080 x 5 =	0.40%	----	0.070 - 0.090
7.	Subcon :	0.100 x 25 =	2.50%	----	0.090 - 0.120
	TOTAL :	=	8.10%		

Statements for profit calculations:

1. Risk (0.80): This project has a some degree of risk and it includes dealing with the traffic at the channel.

2. Difficulty (0.080): The difficulty of this project is tied in with the risk of doing the task and the proximity of the disposal sites and the depth of dredge.

3. Size (0.030): The total cost of all works in this project is in the range of over $10,000,000.

4. Period (0.110): This project may last longer than 24 months.

5. Investment (0.050): Contractor's investment on this project is above average, and with little or no government-furnished property.

6. Assistance by Government (0.080): There is no government assistance or government-owned property will be used in accomplishing this project.

7. Subcontracting (0.100): This project will probably involved some subcontracting depending on the contractor's capability.

MCACES COST ESTIMATE
UPPER GUADALUPE RIVER FLOOD PROTECTION PROJECT
100 YEAR PROTECTION (NED) PLAN
SUMMARY OF FIRST COST FULLY FUNDED

ACCOUNT CODE	ITEM	ESTIMATED COST SUBTOTAL OCT 97 $	CONTINGENCY	ESTIMATED COST TOTAL OCT 97 $	FULLY FUNDED ESTIMATE ESCALATION MDPT CONST	FULLY FUNDED ESTIMATE TOTAL $	MDPT CONST
	FEDERAL COST	Price Level as of Oct. 97					
01------	LANDS & DAMAGES	$397,500	$59,602	$457,102	$88,723	$545,825	11/03
14------	RECREATION PARKS	$838,232	$162,198	$1,000,430	$194,183	$1,194,613	11/03
15------	CIVIL WORKS	$63,014,025	$11,878,556	$74,892,581	$14,536,650	$89,429,231	11/03
30------	PLANNING, ENGINEERING & DESIGN	$2,625,000	$0	$2,625,000	$509,513	$3,134,513	11/03
31------	CONSTRUCTION MANAGEMENT	$851,000	$0	$851,000	$165,179	$1,016,179	11/03
FEDERAL COST, TOTAL		$67,725,757	$12,100,356	$79,826,113	$15,494,249	$95,320,362	
	NON-FEDERAL COST	Price Level as of Oct. 97					
01------	LANDS & DAMAGES	$42,885,500	$6,150,800	$49,036,300	$9,517,946	$58,554,246	11/03
14------	RECREATION PARKS	$838,232	$162,198	$1,000,430	$194,183	$1,194,613	11/03
------	CIVIL WORKS	$3,041,545	$419,155	$3,460,700	$671,722	$4,132,422	11/03
30------	PLANNING, ENGINEERING & DESIGN	$875,000	$0	$875,000	$169,838	$1,044,838	11/03
31------	CONSTRUCTION MANAGEMENT	$682,000	$0	$682,000	$132,376	$814,376	11/03
NON-FEDERAL COST, TOTAL		$48,322,277	$6,732,153	$55,054,430	$10,686,065	$65,740,495	
	TOTAL PROJECT COST	Price Level as of Oct. 97					
01------	LANDS & DAMAGES	$43,283,000	$6,210,402	$49,493,402	$9,606,669	$59,100,071	11/03
14------	RECREATION PARKS	$1,676,464	$324,396	$2,000,860	$388,367	$2,389,227	11/03
15------	CIVIL WORKS	$66,055,570	$12,297,711	$78,353,281	$15,208,372	$93,561,653	11/03
30------	PLANNING, ENGINEERING & DESIGN	$3,500,000	$0	$3,500,000	$679,350	$4,179,350	11/03
31------	CONSTRUCTION MANAGEMENT	$1,533,000	$0	$1,533,000	$297,555	$1,830,555	11/03
PROJECT COST, TOTAL		$116,048,034	$18,832,509	$134,880,543	$26,180,313	$161,060,856	

MCACES COST ESTIMATE
UPPER GUADALUPE RIVER FLODD PROTECTION PROJECT
100-YEAR PROTECTION (NED) PLAN
DETAIL SUMMARY OF FIRST COST

COUN DE	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY	CONTING. %	REASON
	Price Level as of Oct. 97							
TOTAL PROJECT COST								
01-----	LANDS & DAMAGES							
	Land Payments, Real Estate Documents, Appraisal Documents, ROW, LERRD, Etc.	1	JOB	LS	$43,283,000	$6,210,402		1*
					========	=======		
	Subtotal, Lands & Damages Costs:				$43,283,000			
	Contingencies					$6,210,402		
					========	=======		
01-----	LANDS & DAMAGES, TOTAL					$49,493,402		
14-----	- RECREATION PARKS							
	- Federal & Non-Federal	1	JOB	LS	$1,676,464	$324,396		4*
15-----	- FLOODWAY CONTROL & DIVERSION STRUCTURES							
	- Federal & Non-Federal	1	JOB	LS	$66,055,570	$12,297,711		4*
30-----	PLANNING, ENGINEERING & DESIGN							
	E&D	1	JOB	LS	$3,500,000	$0		
31-----	CONSTRUCTION MANAGEMENT (S&I)							
	SIOH	1	JOB	LS	$1,533,000	$0		
					========	=======		
	Subtotal Project Cost:				$116,048,034			
	Contingencies @ average of	16.2 %				$18,832,509		
					========	=======		
	PROJECT COST, TOTAL					$134,880,543		

MCACES COST ESTIMATE
UPPER GUADALUPE RIVER FLODD PROTECTION PROJECT
100-YEAR PROTECTION (NED) PLAN
DETAIL SUMMARY OF FIRST COST

COUN CODE	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY	CONTING. %	REASON
	Price Level as of Oct. 97							
	FEDERAL COSTS							
01-----	LANDS & DAMAGES							
	Land Payments, Real Estate Documents, Appraisal Documents, ROW, LERRD, Etc.	1	JOB	LS	$397,500	$59,602	15%	1*
	Subtotal, Lands & Damages Costs:				$397,500			
	Contingencies					$59,602		
01-----	LANDS & DAMAGES, TOTAL					$457,102		
14-----	- RECREATION PARKS							
1401--- --	- Mob/Demob	1	JOB	LS	$22,391	$4,333	20.0	2*, 3*, 4*
1402--- --	- Reach 7A	1	JOB	LS	$78,959	$15,279	20.0	2*, 3*, 4*
1403--- --	- Reach 7B & 8	1	JOB	LS	$524,630	$101,515	20.0	2*, 3*, 4*
1404--- --	- Reach 10C	1	JOB	LS	$18,336	$3,548	20.0	2*, 3*, 4*
1405--- --	- Reach 11	1	JOB	LS	$146,939	$28,433	20.0	2*, 3*, 4*
1406--- --	- Reach 12	1	JOB	LS	$46,977	$9,090	20.0	2*, 3*, 4*
	Subtotal:				$838,232			
	Contingencies @ average of	19.4 %				$162,198		
14-----	RECREATION PARKS Sub-Total					$1,000,430		
15-----	FLOODWAY CONTROL & DIVERSION STRUCTURES							
1502--- --	- Reach 7: D/S SPTC to D/S UPRR	1	JOB	LS	$14,519,153	$2,752,511	20.0	2*, 3*, 4*
1503--- --	- Reach 8: D/S UPRR to D/S Willow Glenn	1	JOB	LS	$5,169,491	$995,945	20.0	2*, 3*, 4*
1504--- --	- Reach 9: D/S Willow Glenn to Curtner Ave.	1	JOB	LS	$10,806,559	$2,066,820	20.0	2*, 3*, 4*
1505--- --	- Reach 10A: Curtner Ave. to S. Almaden Expw'y.	1	JOB	LS	$4,509,038	$869,977	20.0	2*, 3*, 4*
06--- --	- Reach 10B: S. Almaden Expw'y. to N. Almaden Expw'y	1	JOB	LS	$570,858	$60,053	20.0	2*, 3*, 4*
07--- --	- Reach 10C & 10D: Stream Gage Station	1	JOB	LS	$5,693,702	$1,069,832	20.0	2*, 3*, 4*
1508--- --	- Reach 11: Cap'tl Expw'y to Branham Lane	1	JOB	LS	$4,811,369	$882,643	20.0	2*, 3*, 4*
1509--- --	- Reach 12: Chynoweth to Blossom Hill Rd.	1	JOB	LS	$848,640	$68,311	20.0	2*, 3*, 4*
1510--- --	- Canoas Creek: U/S Almaden to 115' Night'gale	1	JOB	LS	$1,355,955	$262,375	20.0	2*, 3*, 4*
1511--- --	- Ross Creek: Almaden Expw'y. to Jarvis Ave.	1	JOB	LS	$4,865,738	$941,513	20.0	2*, 3*, 4*
1512--- --	- Excess Excavated Soil Disposal	1	JOB	LS	$9,863,522	$1,908,576	20.0	2*, 3*, 4*
	Subtotal:				$63,014,025			
	Contingencies @ average of	18.9 %				$11,878,556		
15-----	- FLOODWAY CONTROL & DIVERSION Sub-Total					$74,892,581		
30-----	- PLANNING, ENGINEERING & DESIGN							
	- E&D	1	JOB	LS	$2,625,000	$0		
31-----	- CONSTRUCTION MANAGEMENT (S&I)							
	- SIOH	1	JOB	LS	$851,000	$0		
	Subtotal, Federal Costs:				$67,725,757			
	Contingencies @ average of	17.9 %				$12,100,356		
	FEDERAL COST, TOTAL					$79,826,113		

MCACES COST ESTIMATE
UPPER GUADALUPE RIVER FLODD PROTECTION PROJECT
100-YEAR PROTECTION (NED) PLAN
DETAIL SUMMARY OF FIRST COST

ACCOUN DE	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY	CONTING. %	REASON
	Price Level as of Oct. 97							
	NON-FEDERAL COSTS							
01——	- LANDS & DAMAGES							
	- Land Payments, Real Estate Documents, - Appraisal Documents, ROW, LERRD, Etc.	1	JOB	LS	$42,885,500	$6,150,800	14%	1*
	Subtotal, Lands & Damages Costs:				$42,885,500			
	Contingencies					$6,150,800		
01——	LANDS & DAMAGES, TOTAL					$49,036,300		
14——	- RECREATION PARKS							
1401——	- - Mob/Demob	1	JOB	LS	$22,391	$4,333	20.0	2*, 3*, 4*
1402——	- - Reach 7A	1	JOB	LS	$78,959	$15,279	20.0	2*, 3*, 4*
1403——	- - Reach 7B & 8	1	JOB	LS	$524,630	$101,515	20.0	2*, 3*, 4*
1404——	- - Reach 10C	1	JOB	LS	$18,336	$3,548	20.0	2*, 3*, 4*
1405——	- - Reach 11	1	JOB	LS	$146,939	$28,433	20.0	2*, 3*, 4*
1406——	- - Reach 12	1	JOB	LS	$46,977	$9,090	20.0	2*, 3*, 4*
	Subtotal:				$838,232			
	Contingencies @ average of	19.4				$162,198		
14——	RECREATION PARKS					$1,000,430		
15——	- FLOODWAY CONTROL & DIVERSION STRUCTURES							
1501——	- - Reach 10C & 10D: Stream Gage Station	1	JOB	LS	$3,041,545	$419,155	20.0	2*, 3*, 4*
	Subtotal:				$3,041,545			
	Contingencies @ average of	13.8 %				$419,155		
5——	CIVIL WORKS , TOTAL					$3,460,700		
——	PLANNING, ENGINEERING & DESIGN E&D	1	JOB	LS	$875,000	$0		
31——	CONSTRUCTION MANAGEMENT	1	JOB	LS	$682,000	$0		
	Subtotal, Non-Federal Costs:				$48,322,277			
	Contingencies @ average of	13.9				$6,732,153		
	NON-FEDERAL COST, TOTAL					$55,054,430		

Contingency Footnotes:

1* Real Estate costs was provided by Sacramento District.

2* Mobilization and demobilization costs would be affected by the availability and origin of plant and equipment.

3* MOB/DEMOB cost is built-in with individual task.

4* Construction of the main flood protection project will be affected by the final design of the recommended plan for the project.

PART II

HYDROLOGY & HYDRAULICS ANALYSIS

SUMMARY OF HYDROLOGY AND HYDRAULICS

1.0 Problems and Opportunities

1.1 Flooding Problems

The threat of significant flooding is a water resource related problem identified in the study area. Recorded or known flooding from the Guadalupe River dates back to the early 1800's. Newspaper accounts described notable floods which occurred in 1862, 1895, 1911 and 1917, but streamflow data has only been compiled since 1930.

Impacts due to flooding have intensified since World War II as the valley's land use changed from agricultural to residential and industrial. Fourteen floods have occurred since World War II. Flooding in 1955 was the worst recorded in history, inundating 8,300 acres in the Guadalupe River drainage basin and causing $18 million in damages (1995 dollars).

The study area has been flooded on six occasions in the past 15 years. The Guadalupe River overtopped its banks upstream of Alma Avenue during floods in February 1980, March 1982, January 1983, February 1986, January 1995, and March 1995. Canoas and Ross Creeks have also overtopped their banks in the last 15 years.

In 1982, flooding spread to about 15 acres that included the areas bounded by the Elks Lodge on Alma Street, West Virginia Street, the Southern Pacific Railroad and McLellan Avenue. The Elks Lodge reportedly had about one foot of water inside, while the Alma Street and Willow Street under crossings, beneath the Southern Pacific railroad bridges, experienced about ten feet of flooding. The March 1982 flooding caused damage to twenty homes and five businesses, which totaled approximately $410,000 (1982 dollars).

In January 1983, the Guadalupe River flooded between Union Pacific Railroad and West Virginia Street and inundated about 60 acres. The peak flow recorded was 8,400 cfs at Almaden Expressway, approximately a 17-year flood event. The Governor of California issued a State of Emergency Declaration for Santa Clara County, which was followed by a Declaration of Major Disaster for Public Assistance by the President of the United States. The January 1983 event was the most severe during the 1980s.

In January 1995, the Guadalupe River overtopped its banks from a point 500 feet upstream of the Southern Pacific railroad tracks at the border of the Elks Lodge parking lot to Interstate 280 (see Plate 1). Flooding also occurred near the confluence with Los Gatos Creek. Ross Creek and Canoas Creek also overtopped their banks and flooded residential areas.

Later the same year in March the Guadalupe River overtopped its banks between Interstate 280 and Willow Glen Way. There was 2 to 3 feet of flooding in the Elks Lodge and flood waters entered several homes and commercial buildings (see Plate 2).

High floods have also caused extensive erosion damage, sediment deposition and debris accumulation. One retaining wall also failed along Old Almaden Road as a result of flooding in the 1980s.

Previous channel modifications and erosion protection activities have extensively altered the river channel within the study area. In the late nineteenth century, the section between Willow Street and Willow Glen Way was realigned to reduce flood damages. The confluence of Canoas Creek and Guadalupe River was moved upstream from its original location (near Auzerias Avenue in downtown San Jose) to its present location upstream of Curtner Avenue. Gravel excavation operations resulted in channel widening between Branham Lane and Blossom Hill Road. Construction of the Almaden Expressway resulted in realignment and widening along a 3,000-foot section of the channel, upstream of Curtner Avenue. Bridge crossings, erosion protection features and the adjacent development have also modified other channel areas.

2.0 Drainage Basin

2.1 Regional Drainage System

The two principal drainage systems for central and western Santa Clara Valley are Guadalupe River and Coyote Creek. Elevations in the Guadalupe River basin ranges from sea level at the San Francisco Bay to about to over 3,000 feet above mean sea level. A summary of elevation ranges is provided in Table 1.

TABLE 1. ELEVATION RANGES & DRAINAGE DISTRIBUTION

Elevation Range (feet, MSL)	Drainage Area (square miles)	Percent of Total Area
0-500	177	34
500-1,000	72	14
1,000-2,000	165	32
2,000-3,000	100	19
over 3,000	6	1
TOTAL	520	100

2.2 Guadalupe River Basin

The Guadalupe River watershed is bounded on the south and southwest by the east side of the Santa Cruz Mountains, on the west by the San Tomas and Saratoga Creeks basin, on the north by the San Francisco Bay and on the east by the Coyote Creek basin. The Guadalupe River is the second largest stream in Santa Clara County discharging into the South San Francisco Bay, draining an area of approximately 170 square miles.

The Guadalupe River corridor passes through a region of the valley that is predominantly residential and commercial. The river flows from south to north for approximately 20 miles before emptying into San Francisco Bay at Alviso Slough. Its upstream terminus is located one-quarter mile south of Blossom Hill Road, at the confluence of Guadalupe Creek and Alamitos Creek. The upstream 5-1/2 miles of the reach comprises the study area. The remaining downstream reaches of the river bisect the redeveloped downtown business district of San Jose and traverse an industrial and publicly zoned area the San Jose International Airport before emptying into Alviso Slough (The Corps of Engineers previously conducted a study of the downtown reaches, as reported in 1985).

2.3 Description of Major Tributaries

The Guadalupe River has five major tributaries as indicated by Table 2 below.

TABLE 2. MAJOR TRIBUTARIES TO THE GUADALUPE RIVER

Tributary	Drainage Area (square miles)
Los Gatos Creek	52
Guadalupe Creek	13
Alamitos Creek	38
Canoas Creek	19
Ross Creek	10

All tributaries, except for Los Gatos Creek, contribute flows to the upper Guadalupe River reach. Ross Creek merges with the river near Branham Lane. Canoas Creek joins the river near Curtner Avenue. Guadalupe Creek and Alamitos Creek merge with the Guadalupe River near Coleman Road.

2.4 Local Drainage

The city of San Jose is responsible for addressing local drainage problems through construction of curbs, gutters, inlets and storm drains. The local drainage system collects surface runoff and directs it into the nearby river channel and major tributary creeks.

3.0 Existing Water Resources Development Facilities

Santa Clara County has extensive water resources development, including small and large reservoirs for recreation, irrigation, water supply and conservation. The Santa Clara Valley Water District (SCVWD) has constructed dams and reservoirs since 1933, and now operates ten facilities, which are listed along with their storage capacity in Table 3 below. Elsman reservoir is owned and operated by the San Jose Water company. Three of the reservoirs are located upstream of the study area on tributaries to the Guadalupe River: Calero Reservoir, Guadalupe Reservoir and Almaden Reservoir. The reservoirs do not serve a flood control purpose, but do provide incidental flood control.

TABLE 3. SANTA CLARA COUNTY RESERVOIRS CAPACITIES

Reservoir	Storage Capacity (acre-ft)
Almaden	1,780
Anderson	91,280
Calero	10,160
Chesbro	8,086
Coyote	23,700
Elsman	6,200
Guadalupe	3,740
Lexington	20,210
Stevens Creek	3,600
Uvas	10,000
Vasona	410

Numerous flood control structural measures have been or are being constructed on the Guadalupe River. The channel capacity between San Francisco Bay and Highway 101 was increased to convey a 100-year flood event, by widening the river corridor and constructing levees. The reach between Highway 101 and Interstate 880 has capacity for 100-year flood flow, but does not have adequate freeboard to meet the standards of the Federal Emergency Management Agency's Flood Insurance Program. The SCVWD is developing plans for structural improvements to meet these standards.

On March 30, 1992, the SCVWD and the Corps of Engineers Sacramento District Office signed a local cooperation agreement to implement the downtown Guadalupe River Flood Control Project. The project consists of channel modifications between Interstate 880 and Interstate 280 (immediately downstream of the current study area) and includes recreation features. The project is expected to be constructed at a total cost of $138 million. The SCVWD, the San Jose Redevelopment Agency and the State of California will share approximately $78 million of the total cost, as co-sponsors of the project. Construction began in late September 1992, and is scheduled for completion in fall 1998.

The SCVWD is undertaking two local flood control projects within the study area, independently and without Federal contribution. The first local project is a bypass channel adjacent to the east bank of the Guadalupe River alignment from Interstate 280 to Willow Street. The bypass channel would tie into the downtown Guadalupe River project at Interstate 280. These plans are currently being developed, and expected to be constructed by 1998. In 1997, flood control planning studies will also begin for a second local project, along the Canoas Creek tributary. Project construction is expected to begin after 2000.

4.0 Climate

San Jose's climate is typical of the San Francisco Bay area. Summers are dry and warm, while winters are mild and moderately wet. Differential heating between the sea breezes and interior valleys dominate the summer weather. Storms from the north Pacific influence the winter weather, producing virtually all the rainfall in the area.

Extreme temperatures range from a minimum in the 20s to a maximum of over 100 degrees Fahrenheit. An average low temperature of 49 degrees occurs in January, while an average high temperature of 81 degrees occurs in July. The sea breezes and topography of the area influence prevailing wind direction. Winds come most frequently from the northwest. The surrounding mountains provide shelter to the area which results in wind velocities that are generally lower than in other areas surrounding the San Francisco Bay.

4.1 Precipitation

Precipitation data is collected from numerous stations within the study area. Data from the Los Gatos, San Jose and Santa Clara University stations were used to develop the distribution of monthly average precipitation in the basin. The periods of record for the three stations are 92, 103, and 95 years respectively.

Normal annual precipitation (NAP) varies from 14 inches near the San Francisco Bay to 50 inches near the crest of the Santa Cruz Mountains. The normal annual precipitation in the Guadalupe River basin is 26 inches. Table 4 tabulates the distribution of monthly average precipitation. Ninety percent of the rainfall occurs during the winter, in the six-month period of November through April. January is normally the wettest month.

TABLE 4. MONTHLY AVERAGE PRECIPITATION

Month	Monthly Rainfall Percent of NAP	Month	Monthly Rainfall Percent of NAP
Jan	21.1	Jul	0.1
Feb	16.5	Aug	0.2
Mar	14.0	Sep	0.9
Apr	8.7	Oct	4.7
May	2.4	Nov	11.9
Jun	0.4	Dec	19.1

4.2 Runoff

Runoff varies significantly and has been highly modified by storage and diversion facilities. Natural average annual runoff was estimated to be 35,500 acre-feet, from gaging station records on the Guadalupe River in San Jose for the period of 1931 to 1960. The record shows precipitation ranging from zero in 1931 to over 123,000 acre-feet in 1938, which is believed to be wettest year of record. Monthly average runoff is shown in table 5.

TABLE 5. MONTHLY AVERAGE RUNOFF

Month	Monthly Runoff*	Month	Monthly Runoff*
Jan	19.8	Jul	0
Feb	32.6	Aug	0
Mar	20.5	Sep	0
Apr	10.9	Oct	0
May	0.3	Nov	1.0
Jun	0	Dec	14.9

* Percent of average annual runoff.

4.3 Flow-Frequency Relationships

The District's 1977 Hydrologic Engineering Office Report for the upper Guadalupe River basin serves as the basis for flow-frequency relationships for this study. Results contained in the Hydrologic Engineering Report were confirmed by the Sacramento District Office in the General Design Memorandum of December 1991 for the downtown Guadalupe River project. The report is available at the District's office. Channel design discharges and their frequencies are shown on page 12 in section 7.7 of this report.

5.0 Previous Hydrologic Studies

The 1977 Corps of Engineers Hydrologic Engineering Report serves as the basis for this feasibility study. The report includes discharge versus frequency relationships and the Standard Project Flood discharge for the Guadalupe River, Coyote Creek and Ross Creek basins. The Sacramento District also adopted the hydrologic study findings for the downtown Guadalupe River project, which is currently under construction.

A second hydrologic engineering office report (published in December 1991 and revised in April 1992) develops hydrology for the Canoas Creek basin and assesses factors which could cause the upper Guadalupe River basin hydrology to change. The conclusion from this report was that any expected basin changes would have minimal effect on the Canoas Creek basin hydrology. Specifically, the study investigated peak discharges for expected future conditions on Canoas Creek; updated spillway discharge versus frequency relationships for upstream reservoirs to reflect the flow conditions subsequent to earlier San Francisco District and Sacramento District study findings; and impacts on downstream discharges due to operational changes of Calero Reservoir (resulting from the importation of water from San Luis Reservoir).

SUMMARY OF HYDRAULICS

6.0 Description of Existing Floodplains

Under existing conditions, the 20-year discharge will overtop both the east and the west banks of the Guadalupe river at various locations from Interstate 280 to Willow Glen Way. Also Ross Creek and Canoas Creek overtop their banks during the 20-year flood event and flow into the Guadalupe River floodplain. Plate 3 delineates the extent of the 20-year floodplain.

The 50-year flood will overtop both the east and west banks of the Guadalupe River at various locations from Interstate 280 to Willow Glen Way. The 50-year flood will overtop the east bank of the Guadalupe River upstream of Branham Lane resulting in street flooding. Flooding will also overtop the banks of Ross and Canoas Creeks. Plate 4 delineates the extent of the 50-year floodplain. The 100-year and 500-year floodplains are similar in shape to the 50-year floodplain with greater floodplain depths and areas. Plates 5 and 6 delineate the 100- and 500-year floodplains.

7.0 Design Considerations

7.1 Existing Conditions

Under current conditions the Guadalupe River's flow capacity within the study area varies from as little as a 6-year event to as much as a 100-year event. Existing bridges would safely (i.e. flows unobstructed by their low cords) allow the passage of similar flows (7- to 100-year flows).

The preceding statements are based on Corps', not FEMA, criteria and assume that the local sponsor has completed construction of proposed improvements (to a 100-year level) from Interstate 280 to Willow Street. Tables 6 and 7 below list the flow capacity for the numerous river reaches and bridges in the study area. See Figure 1 for reach locations.

TABLE 6. EXISTING CAPACITIES BY REACH

Reach	Capacity (cfs)	Exceedance Interval (Years)
7	6,000	6.5
8	8,000	15
9	12,000	60
10	11,000	48[1] 55[2]
11	10,000	55[3] 60[4]
12	8,000	37
Ross Creek	930	5.5
Canoas Creek	1,900	9

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[1] Downstream of Canoas Creek

[2] Upstream of Canoas Creek

[3] Downstream of Ross Creek

[4] Upstream of Ross Creek

TABLE 7. EXISTING CAPACITIES BY BRIDGE

Bridge	Capacity (cfs)	Exceedance Interval (Years)
Virginia Street	10,000	33
SPRR bridge	11,458	50
Willow Street	6,420	7-8
Alma Street	6,300	7-8
UPRR	11,300	50
Willow Glen Way	11,630	53
Malone Road	12,000	55
Curtner Ave	11,340	50
S. Almaden Exprwy- Canoas	10,000	33
N. Almaden Exprwy	21,000	100
Hillsdale Ave	9,680	50
Capitol Exprwy	8,200	33
Branham Lane	7,200	25

7.2 Manning's "n" Values

For existing conditions the natural channel roughness was estimated to have a Manning's "n" value of 0.045 for its entire length. The selected "n" value was input into the existing conditions HEC-2 model, which produced water surface elevations, channel capacities and overtopping discharges which compared well with historical events. Changing the channel "n" value from reach to reach would not appreciably change the calculated floodplains.

7.3 Topographic Data

Topographic data used for floodplain analysis was obtained from aerial photographic maps (1-inch = 200 feet scale) with 2-foot contours and spot elevations. These maps are based on aerial surveys of the floodplain conducted in November 1990 by Aero-Geodetic Corporation.

7.4 Guadalupe River Floodplains

Floodplains under current condition were developed for the 20-, 50-, 100- and 500-year flood events. They are shown on Plates 3, 4, 5 and 6. The current floodplains are based on the following methodology/criteria:

1. Overtopping discharges at low capacity channel cross-sections were calculated for the existing channel conditions using the HEC-2 split flow option, and were then routed through the floodplain for each flood event.

2. Floodplain depths are based on normal depth calculations. Normal depth calculations included Manning's "n" values of 0.02 to 0.10.

3. Normal depth cross-sections extend across the entire floodplain width where flood flows can move without excessive interference. The cross-sections were developed from the 1" = 200' scale topographic maps.

4. Normal depth cross-sections are limited to streets parallel to the direction of flow when flood flows are blocked by homes, buildings and fences, etc. Normal depths calculated for street flow are then extended across the floodplain.

5. Ponding was assumed in low areas.

7.5 Low Capacity Channel sections

The existing upper Guadalupe River channel has relatively low capacity at specific locations. The locations, discharges and frequencies associated with the low-capacity areas are listed below (see Plate 7 for economic areas).

Summary of Low Capacity Cross-sections
Used for Economics and Risk Analysis

Economic Area	Station	Discharge (cfs)	Frequency (years)
1	746+00	6,000	6.5
2	Ross Creek	930	5.5
3	777+00	6,100	7
4	Canoas Creek	1,900	9
5	961+00	8,000	37

7.6 Design Limitations

Mitigation - The Guadalupe River channel contains a valuable riparian corridor; therefore, steps will be taken to minimize impacts. Design methods used to minimize impacts to the riparian corridor include limiting channel excavation to one bank for any reach when possible. With the Willow Glen and Valley View plans, the channel cross-sections will be excavated so that there is a bench 3-10 feet above the low flow channel invert, floodwalls will be used along channel reaches where interior drainage is not a problem. With the Bypass Channel Plan, a bypass channel will be constructed to run parallel to the existing channel so that parts of the riparian corridor remain completely untouched.

The above channel configuration will minimize impacts to the riparian habitat as well as maintain the efficiency of the sediment transport capability of the existing channel. The channel cross-sections will be designed with a high Manning's "n" (up to 0.070) so that riparian habitat removed during construction will be allowed to re-establish in the river channel. All plans will require the replacement or the modification of several bridges.

Real Estate - Real estate for both project construction and mitigation is very expensive; therefore, real estate acquisition must be minimized. Real estate restrictions will require the construction of steep banks and extensive bank protection in some areas.

Plans Studied - The NED analysis is based on the evaluation of the Willow Glen, Valley View, and Bypass Channel designs. Each plan was designed to contain its respective level of protections with one foot of uncertainty in the design water surface elevation. Discharges are based on the results obtained in the Hydrologic Engineering Office Report, Guadalupe River and Coyote Creek, Santa Clara County, California, June 1977.

7.7 Channel Design Discharges

The channel design flows tabulated below are based on future hydrologic conditions (the year 2010). Detailed hydrology is available in the Hydrologic Engineering Office Report for the Guadalupe River and Coyote Creek, Santa Clara County, California, June 1977.

Channel Design Discharges
Guadalupe River, San Jose, California
Future Flows
(Existing Flows shown in parentheses)

Location/Event	Drainage Area sq mi	20-yr	50-yr	100-yr	500-yr
Guadalupe River downstream of Canoas Creek	81	9,000 (9,000)	11,200 (11,200)	14,600 (13,200)	21,800 (18,000)
Guadalupe River downstream of Ross Creek	63	7,300 (7,100)	9,700 (9,700)	12,400 (11,800)	19,000 (17,700)
Guadalupe River downstream of Blossom Hill Road	53	6,300 (6,300)	9,100 (9,100)	11,400 (11,400)	17,800 (17,000)
Canoas Creek at Guadalupe River	18	2,500 (2,500)	3,000 (3,000)	3,300 (3,300)	3,600 (3,600)
Ross Creek at Guadalupe River	10	1,550 (1,550)	1,950 (1,950)	2,350 (2,350)	3,100 (3,100)

7.8 Manning's "n" Values for Project Conditions

A composite "n" value for the channel cross-section was computed based on the table of roughness values shown below.

CHANNEL ROUGHNESS FACTORS

Type of Cover	Manning's n - value
Representative open channel cross-section (combination of cover types - composite "n" value)	0.055 - 0.062
Box culvert	0.025
Rip-rap-lined channel slope	0.045
Grass or gravel-lined bench	0.030
Instream riparian habitat	0.110
Natural channel bottom	0.040

8.0 Plans of Improvement

Three plans of improvement were evaluated: the Willow Glen Plan, Valley View plan and Bypass Channel plan. These plans feature floodwalls, channel widening, a channel bypass and bridge modifications/replacement as means to alter the upper Guadalupe River from Interstate 280 upstream to Blossom Hill Road, a distance of about 5.5 miles. Improvements on Ross Creek will be constructed from Guadalupe River to Jarvis Avenue. For all designs (Willow Glen, Valley View, and Bypass Channel) it is assumed that a project with a 20-year level of protection will be constructed on Canoas Creek.

8.1 Willow Glen Plan

Following construction of this plan the upper Guadalupe River would contain up to a 20-year discharge, or 9,000 cfs downstream of Canoas Creek. Channel widening will be limited to one bank of the river to preserve the existing riparian habitat. Above Canoas Creek the Guadalupe River has a natural level of protection of 37- to 60-years. Ross Creek would contain up to a 20-year discharge from the Guadalupe River to Jarvis Avenue. A plan view is shown on Plate 8. For discussion purposes the study reach is separated into subreaches and described accordingly.

Interstate 280 to SPRR: The local sponsor would construct a 2,500-foot long bypass channel parallel to the river on the east side. The bypass would be trapezoidal in shape and lined with riprap. The combined capacity of the bypass and existing river channel would equal a 100-year discharge of 14,600 cfs.

SPRR to State Highway 87: This portion of the project will join the local sponsor's improvements at the SPRR bridge. Channel improvements will include widening the channel to create a channel width of 110 feet, 3 feet above the channel invert (See Figure 2, 50-year level of protection as a reference). Side slopes would be 1V on 1.5H where possible with bank protection as needed. Channel roughness would reflect a composite Manning's "n" on the order of 0.062.

State Highway 87 to UPRR Bridge: Improvements for this project section will include a channel width of 80 feet, 3 feet above the channel invert (See Figure 2, 50-year level of protection as a reference), and a floodwall 2-4 feet high on the east bank between the Alma street and UPRR bridges. The Alma Street bridge and the Willow Street bridge will require modifications or replacement. The design composite Manning's "n" value is 0.062.

UPRR Bridge to Willow Glen Way: End of work on Guadalupe River improvements for this project section will include floodwalls on both banks 1 to 3 feet high and improvements to the UPRR bridge. The design composite Manning's "n" value is 0.045.

Willow Glen Way to Blossom Hill Road: The Guadalupe River channel and bridge openings have at least 20-year capacity from Willow Glen Way to Blossom Hill Road. The assumed composite Manning's "n" value for this section is 0.045.

8.1.1 Canoas Creek

Flooding on Canoas Creek results from two different mechanisms; the runoff contribution from Canoas Creek's watershed and backwater flows from the Guadalupe River. The backwater effects are limited to the lower reach of Canoas Creek. To address the runoff contribution would require an extensive study and may require improvements on the same order of magnitude as the main stem, the SCVWD indicated that they intend to manage the peak runoff flows from the Canoas Creek watershed through local measures. Thus, for plan formulation purposes, improvements to Canoas Creek were limited to backwater effects.

Flooding is a recurrent problem in the downstream reach of Canoas Creek. The downstream portion of the creek has a capacity limited to approximately a 9-year event. It is estimated that the capacity of the upstream portion of the creek is limited to flows associated with an approximate 20-year event. Conditions on Canoas Creek were based on the Corps' December 1991 (revised April 1992) Hydrologic Engineering Office Report, Upper Guadalupe River Basin, Santa Clara County, California.

All improvements to alleviate backwater flooding were designed to also contain flows associated with a 20-year event or less from Canoas Creek. Thus, each of the plans described include identical improvements to Canoas Creek. These include the replacement of culverts beneath Almaden Expressway and Nightingale Drive and low floodwalls along both banks of the creek. It was assumed that any additional improvements undertaken to increase the capacity of Canoas Creek beyond an approximate 20-year capacity would be undertaken as a local project.

For the purposes of sizing improvements on the main stem Guadalupe River, it was assumed that the main stem channel directly downstream of the confluence with Canoas Creek would accommodate flows associated with any event from Canoas Creek. This assumption was made to be consistent with the SCVWD's intention to manage peak runoff on Canoas Creek through local measures. Note that because peak flows on Canoas and the main stem are not coincident, the additional peak flows from Canoas are incidental when compared with flows in the main channel.

8.1.2 Ross Creek

To provide a 20-year level of protection on Ross Creek the following improvements will be made. The bottom width will be excavated to 25 feet with 1V:1H side slopes. Floodwalls will be built from Almaden Expressway to a point 300 feet upstream of Cherry Avenue. Interior drainage requirements for the floodwall have not been calculated for the Willow Glen Plan since it is very unlikely to ever be selected as the NED plan. An additional 12' W x 10 'H 210'-long culvert will be added at Almaden Expressway. At Jarvis Avenue a 12'W x 9.5'H culvert will be installed in addition to the existing 12'W x 9.5'H culvert.

8.2 Valley View Plan

The Valley View plan features channel widening on Guadalupe River and Ross Creek, bridge replacement and modification, and floodwalls designed to contain the 50-year discharge. Channel widening will be limited to one bank in most cases to preserve the existing riparian habitat. A plan view of the Valley View plan is shown on Plate 9; typical cross-sections are shown on Figures 2-6. The plan is described below.

Interstate 280 to SPRR: The local sponsor will build a bypass channel parallel to the existing Guadalupe River channel. The combined capacity of the existing river channel and the bypass channel will be equal to a 100-year level of protection.

SPRR to State Highway 87: This section of the project will join the local sponsor's improvements at the SPRR bridge. Improvements for this project section will consist of excavation to construct a bank to channel width of 110 feet, 3 feet above the invert (See Figure 2, 50-year level of protection as a reference). The design composite Manning's "n" value is 0.07.

State Highway 87 to UPRR Bridge: Improvements for this project section will include excavation to provide a bank to bank width of 95 feet, 3 feet above the channel invert (See Figure 2, 50-year level of protection as a reference), and a floodwall 2-4 feet high between the Alma Street bridge and a point just upstream of the UPRR bridge. Also, the Alma Street bridge and the Willow Street bridge will require replacement. The design composite Manning's "n" value is 0.07.

UPRR Bridge to Willow Glen Way: Improvements for this project section will include floodwalls on both banks 1 to 3 feet high, replacing the UPRR bridge and replacing the Willow Glen Way bridge. The design composite Manning's "n" value is 0.045.

Willow Glen Way to Curtner Avenue: This section of the Guadalupe River and the bridge openings within it have at least a 50-year capacity. The design composite Manning's "n" value is 0.045.

Curtner Avenue to Almaden Expressway (southbound): Improvements for this project section will include excavation with a bank to bank width of 65-75 feet, 3 feet above the channel invert (this configuration will provide a low flow channel and a bench 3 feet above the channel invert). The design composite Manning's "n" value is 0.055

Almaden Expressway (southbound) to 400 feet downstream of Foxworthy Avenue: This section of the Guadalupe River and the bridge openings within it have at least a 50-year capacity. The design composite Manning's "n" value is 0.045.

400 feet downstream of Foxworthy Avenue to Capitol Expressway: Improvements for this project section will include a channel with bank to bank width of 85 feet, 3 feet above the channel invert (See Figure 2, 50-year level of protection as a reference). The Hillsdale Avenue bridge will be replaced. The design composite Manning's "n" value is 0.055.

Capitol Expressway to Carrie Lee Way: This section of the Guadalupe River and the bridge openings within it have at least a 50-year capacity. The design composite Manning's "n" value is 0.045.

Carrie Lee Way to Branham Lane: Improvements for this project section will include a channel with bank to bank width of 85 feet, 3 feet above the channel invert (See Figure 2, 50-year level of protection as a reference). The design composite Manning's "n" value is 0.055.

Branham Lane to Blossom Hill Road: This section of the Guadalupe River and the bridge openings within it have at least a 50-year capacity. The design composite Manning's "n" value is 0.040.

8.2.1 Ross Creek

Improvements to Ross Creek will include new culverts and channel widening. The excavated channel bottom width will be 27 feet from Almaden Expressway to a point 750 feet upstream of Jarvis Avenue. An additional 12'W x 10'H x 210'-long box culvert will be installed next to the existing 10H x 12W x 210'-long culvert at Almaden Expressway. At Jarvis Avenue a 12'W x 9.5'H culvert will be installed in addition to the existing 12' W x 9.5'H culvert. Intermittent floodwalls will be built from Almaden Expressway to a point 300 feet upstream of Cherry Avenue, and between Jarvis Avenue to a point 600 feet upstream of Jarvis.

8.2.2 Canoas Creek

Canoas Creek will be improved by replacing the culverts beneath Almaden Expressway and Nightingale Drive and providing low floodwalls along both banks of the creek.

8.3 Bypass Channel Plan

The Bypass Channel plan features a bypass channel, channel widening, levee and floodwalls designed to contain the 100-year discharge on the upper Guadalupe River and Ross Creek. Channel widening will be limited to one bank in most cases to preserve the existing riparian habitat. A plan view of the Bypass Channel Plan is shown on Plate 10. The plan is described below.

Interstate 280 to 500' upstream of Willow Street: Improvements to this project section will include an 18'-deep bypass channel with an 85'-wide floodway and 1:1 side slopes.

500' upstream of Willow Street to Alma Street: Improvements to this project section will consist of a combined natural and bypass channel. 18'-deep bypass with 30'-wide floodway and

1:1 side slopes; the surface elevation of the embankment between the bypass and natural channel would allow transfer of floodwaters between the two alignments during high flows.

Alma Street to UPRR Bridge: A gabion lined bypass channel will be built through the existing Elks Lodge parking lot.

UPRR Bridge to Willow Glen Way: 18'-deep bypass with 85'-wide channel floodway with 1:1 side slopes.

Willow Glen Way to Blossom Hill Road: Improvements to this project section will include channel widening. The bank to bank width of the project will range from 75 feet to 200 feet, 4-10 feet above the invert (See Figure 2, 50-year level of protection as a reference).

8.3.1 Canoas Creek

Canoas Creek will be improved by replacing the culverts beneath Almaden Expressway and Nightingale Drive and providing low floodwalls along both banks of the creek.

8.3.2 Ross Creek

Improvements to Ross Creek will include new culverts and channel widening. The channel bottom width be excavated to 35 feet from Almaden Expressway to a point 750 feet upstream of Jarvis Avenue. The existing 12'W x 9.5'H x 210'-long box culvert at Almaden Expressway will be replaced with a 20'W x 9.5'H x 210'-long culvert. At Jarvis Avenue a 12'W x 9.5'H culvert will be added to the existing 12'W x 9.5'H culvert. Intermittent floodwalls will be built from Almaden Expressway to a point 300 feet upstream of Cherry Avenue, and between Jarvis Avenue to a point 600 feet upstream of Jarvis.

9.0 Residual Flooding

Residual flooding for the Willow Glen Plan, Valley View and the locally preferred plans are shown on Plates 11, 12, and 13.

10.0 Sediment Study

A sediment study for the Guadalupe River was completed by Philip Williams and Associates in February 1996. Results from the HEC-6 models revealed the following:

- Under existing conditions the upper Guadalupe River appears to have a fairly stable sediment transport regime: at any location in the study area, under design flood conditions, reach-averaged deposition produced less than 1.0 foot of bed-elevation increase, and reach-averaged scour produced less than 2.0 feet of bed-elevation drop. The upstream reaches of the model reflected very little bed-elevation change, while the downstream reaches of the model reflected a slight scouring trend.

- The scouring trend observed in the lower reaches of the existing conditions model is indicative of the generally sediment-starved state of the river. The dams upstream of the study area and the increased amount of paved surface due to urbanization in the Guadalupe watershed have both reduced the natural runoff sediment load of the upper Guadalupe River.

- Under the Willow Glen and Valley View Plan conditions, the upper Guadalupe River reflects a slightly more stable sediment transport regime than under existing conditions: design flood conditions produced small reach-averaged bed-elevation changes in the upstream reaches of the study area and reach-averaged scour in the downstream reaches of the study area was reduced from existing condition - especially in the zones where the bypass channel operates. The Willow Glen and Valley View Plans yielded very similar results.

- Under Bypass Channel Plan conditions, the upper Guadalupe River reflected the most stable sediment transport regime modeled: the design flood conditions produced reach-averaged bed-elevation changes that were less than 1.0 feet at any given location in the study area.

- Long-term flow simulations of the upper Guadalupe River - where the effects of a 25-year flow record were modeled for each channel configuration - produced bed-elevation changes that were accentuated from those observed under design flood conditions. However, the locations of deposition and erosion remained very similar to those reflected in the design flood simulations.

- The deposition phenomenon expected to manifest downstream of the bypass diversion weirs was not observed in the Bypass Channel Plan model results, and was observed only on a very minor scale in the Willow Glen and Valley View Plan model results. This diminished deposition was due to the overall sediment-starved condition of the river. Scour downstream of the bypass junctions was also not as extraordinary as expected.

- Sensitivity analysis revealed that the magnitude of resulting bed-elevation changes were most sensitive to the type of sediment transport function and the values of Manning's "n" (as used in the models). However, sensitivity to these parameters was not particularly high, and therefore, the model results are quite reliable. Uncertainty in the magnitude of reach-averaged bed-elevation changes from model results is on the order of 1.5 feet. The location of scour and deposition zones in model results was not significantly sensitive to model parameters.

- Under any of the proposed flood-control alternatives, it appears that channel maintenance requirements would not significantly increase. In fact, less channel maintenance would perhaps be required since each flood-control alternative tends to subdue the patterns of scour and deposition that occur under Existing Conditions. However, it should be noted that the hypothesized patterns of extraordinary deposition and scour associated with the bypass channel junctions - those patterns that did not materialize in model results - could still occur, as HEC-6 is unable to model the complex local transport phenomena associated with bypass inlet and outlet structures. If these patterns of deposition and scour did occur around the bypass junctions, channel maintenance requirements could be significantly greater that under existing river conditions.

For details on the sediment transport study refer to the report "Sediment Transport Modeling Study of the Upper Guadalupe River, April 1996".

11.0 GUADALUPE RIVER RISK BASED ANALYSIS

11.1 Introduction

The risk based analysis (RBA) for upper Guadalupe River was performed using the "at Risk" and the "Lotus 123" computer programs with a "Lotus 123" template developed by staff from the Corps' Hydrologic Engineering Center (HEC) in Davis, California. Adopted methods, assumptions and data used in the analysis to determine with and without project damages are discussed in detail below for final NED analysis and selection.

11.2 Methods, Assumptions and Data

1) The Guadalupe River floodplain is divided into 5 sub-areas, each with one or more flood source. Each of the 5 sub-areas has one flood source that is unique to that sub-area. The 5 sub-areas are shown on Plate 7.
2) Uncertainty in discharge and stage versus frequency curves was determined based on gaged data with a 25-30 year record length, depending upon which gage was used.

3) The water surface stage uncertainty is based on the variability of the channel roughness coefficient, Manning's "n", and the variability of results based on the use of two different sediment transport equations. The roughness coefficients displayed by Table 1 below represent the upper and lower limits of a 90% confidence interval. Also indicated are sediment transport equations by Yang and Meyer-Peter/Muller-Toffeleti which were also used in the hydraulic model to determine a standard deviation of error in water surface stage at low capacity stations in designated economic areas. Note that the effects of channel roughness and sediment transport were determined separately and then combined to arrive at the values standard deviation of error in stage presented in Table 2, under without and with project conditions.

Table 1

"n" Values and Sediment Transport Equations used to Determine Stage Uncertainty

Economic Area	90 % Confidence Interval	Existing Conditions		Project Conditions		
		Channel "n"	Sediment Transport Eq.	Channel "n"	Bypass Channel "n"	Sediment Transport Eq.
1	Lower 5%	.04	Yang	.05	.03	Yang
	Mean	.045		.062	.035	
	Upper 95%	.055	MPMT*	.075	.04	MPMT
2	Lower 5%	.023	Yang	.02	N/A	Yang
	Mean	.028		.025	N/A	
	Upper 95%	.033	MPMT	.030	N/A	MPMT
3	Lower 5%	.04	Yang	.05	.03	Yang
	Mean	.045		.062	.035	
	Upper 95%	.055	MPMT	.070	.04	MPMT
4	Lower 5%	.025	Yang	.025	N/A	Yang
	Mean	.03		.03	N/A	
	Upper 95%	.04	MPMT	.04	N/A	MPMT
5	Lower 5%	.04	Yang	.05	N/A	Yang
	Mean	.045		.055	N/A	
	Upper 95%	.055	MPMT	.075	N/A	MPMT

*Meyer-Peter and Muller-Toffaleti

Table 2
Standard Deviation of Error for Stage (feet)
Based on the Variability of Sediment Deposition and Manning's "n"

Economic Area	Station	Without Project	With Project		
			20-yr	50-yr	100-yr
1	746+00	.75	.8	.9	1.0
2	Ross Creek *	1.0	1.0	1.0	1.0
3	777+00	.7	.85	1.0	1.1
4	Canoas Creek *	1.0	1.0	1.0	1.0
5	961+25	.8	.85	.9	1.0

*The standard deviation error for Ross and Canoas Creeks is based on uncertainties caused by box culverts at the confluence with the Guadalupe River and the lack of detailed sediment analyses for the two creeks.

4) A stage (in channel) vs. discharge rating curve was developed for each sub-area at its unique flood source (the channel reach where the channel bank is first overtopped during a flood event) for both existing and project conditions.

5) Data input into the risk model for each economic area for existing and project conditions.
  A) Discharge Frequency Data; mean, standard deviation, skew, years of record.
  B) Stage vs. Damage Curve.
  C) Stage vs. Discharge Curve.
  D) Uncertainty of Stage.

6) A risk based analysis is run on each channel improvement plan to determine with-project flood damages. These damages are then compared with damages calculated under without-project conditions to determine a reduction in damages or project benefits.

11.3 Risk Based Analysis for the Bypass Channel Plan

The risk and uncertainty analysis was run for cross-sections at points of minimum capacity (design water surface elevations are a maximum of one foot below the top of the channel bank or levee/floodwall). Tables 3 through 7 display the probability that each plan would be exceeded within a given time period at various stations within the project. Tables 4 through 12 display the probability that each plan would *not* be exceeded under various storm conditions at various stations within the project.

Table 3
Annual Performance and Equivalent Long-term Risk
at Station 746+00

Plan Description	Levee/Floodwall /Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Equivalent Long-term Risk (Probability of Exceedence Over the Indicated Time Period)		
			10 Years	20 Years	50 Years
Existing	102.7	0.1568	82	97	99
Willow Glen Plan	102.7	0.0424	35	58	89
Valley View Plan	103.7	0.0212	20	35	67
Bypass Channel	103.7	0.0094	9	17	38

Table 4
Performance of Project Plans Guadalupe River
at Station 777+00

Plan Description	Levee/Floodwall /Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Equivalent Long-term Risk (Probability of Exceedence Over the Indicated Time Period)		
			10 Years	20 Years	50 Years
Existing	111.7	0.1136	70	91	99
Willow Glen Plan	111.7	0.044	36	59	89
Valley View Plan	112.7	0.0114	11	20	44
Bypass Channel Plan	113	0.0032	3	5	13

Table 5
Performance of Project Plans Guadalupe River
at Station 782+00

Plan Description	Levee/Floodwall /Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Equivalent Long-term Risk (Probability of Exceedence Over the Indicated Time Period)		
			10 Years	20 Years	50 Years
Existing	113.5	0.1100	69	90	99
Willow Glen Plan	114.5	0.044	64	59	89
Valley View Plan	114.5	0.011	11	20	43
Bypass Channel Plan	114.5	0.094	9	17	38

Table 6
Performance of Project Plans Guadalupe River
at Station 961+25

Plan Description	Levee/Floodwall /Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Equivalent Long-term Risk (Probability of Exceedence Over the Indicated Time Period)		
			10 Years	20 Years	50 Years
Existing	170.3	0.0576	45	69	95
Willow Glen Plan	N/A	N/A	N/A	N/A	N/A
Valley View Plan	172.5	0.0188	17	32	39
Bypass Channel Plan	172.5	0.0098	9	18	39

Table 7
Performance of Project Plans Guadalupe River
at Ross Creek

Plan Description	Levee/Floodwall/ Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Equivalent Long-term Risk (Probability of Exceedence Over the Indicated Time Period)		
			10 Years	20 Years	50 Years
Existing	167.7	0.1948	88	98	99
Willow Glen Plan	168.0	0.0488	39	63	92
Valley View Plan	168.0	0.0248	22	40	72
Bypass Channel Plan	168.0	0.0088	9	16	36

Table 8
Performance of Project Plans Guadalupe River
at Station 746+00

Plan Description	Levee/Floodwall/ Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Event Performance as a %-Chance Non-Exceedence for Specified Event		
			5%-Chance Event	2%-Chance Event	1%-Chance Event
Existing	102.7	0.1568	5	3	3
Willow Glen Plan	102.7	0.0424	70	38	21
Valley View Plan	103.7	0.0212	88	60	49
Bypass Channel	103.7	0.0094	97	66	77

Table 9
Performance of Project Plans Guadalupe River
at Station 777+00

Plan Description	Levee/Floodwall/ Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Event Performance as a %-Chance Non-Exceedence for Specified Event		
			5%-Chance Event	2%-Chance Event	1%-Chance Event
Existing	111.7	0.1136	22	13	12
Willow Glen Plan	111.7	0.044	70	41	29
Valley View Plan	112.7	0.0114	95	84	74
Bypass Channel	113	0.0032	99	96	91

Table 10
Performance of Project Plans Guadalupe River
at Station 782+00

Plan Description	Levee/Floodwall/ Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Event Performance as a %-Chance Non-Exceedence for Specified Event		
			5%-Chance Event	2%-Chance Event	1%-Chance Event
Existing	113.5	0.1100	22	13	12
Willow Glen Plan	114.5	0.044	70	41	25
Valley View Plan	114.5	0.011	95	80	70
Bypass Channel	114.5	0.0094	99	94	80

Table 11
Performance of Project Plans Guadalupe River
at Station 961+25

Plan Description	Levee/Floodwall/ Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Event Performance as a %-Chance Non-Exceedence for Specified Event		
			5%-Chance Event	2%-Chance Event	1%-Chance Event
Existing	170.3	0.0576	55	19	7
Willow Glen Plan	N/A	N/A	N/A	N/A	N/A
Valley View Plan	172.5	0.0188	91	71	48
Bypass Channel Plan	172.5	0.0098	97	87	74

Table 12
Performance of Project Plans Guadalupe River
at Ross Creek

Plan Description	Levee/Floodwall/ Top of Bank Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Event Performance as a %-Chance Non-Exceedence for Specified Event		
			5%-Chance Event	2%-Chance Event	1%-Chance Event
Existing	167.7	0.1948	31	25	22
Willow Glen Plan	168.0	0.0488	64	46	37
Valley View Plan	168.0	0.0248	83	64	51
Bypass Channel	168.0	0.0088	96	86	76

A floodwall is proposed at station 861+00 for the Bypass Channel Plan. As originally designed, there would be a 74 percent chance that the floodwall would contain floodwaters associated with the 1 percent chance event. In order to increase the reliability to 90 percent, the floodwall was redesigned to be 1.5 feet higher than originally planned. Table 13 displays the performance probabilities for the floodwall as originally designed and as currently designed.

Table 13
Performance of Bypass Channel Plan Floodwall
at Station 861+00

Floodwall Description	Floodwall Elevation	Annual Performance (Expected Annual Probability of Design Being Exceeded)	Equivalent Long-term Risk (Probability of Exceedence Over the Indicated Time Period)			Event Performance as a %-Chance Non-Exceedence for Specified Event		
			10 Years	20 Years	50 Years	5%-Chance Event	2%-Chance Event	1%-Chance Event
Original Design	146.5*	0.0092	9	17	38	97	87	74
Current Design	148.0	0.0042	4	8	18	99	95	90

GEOLOGY & SEISMICITY

UPPER GUADALUPE RIVER
GEOLOGY AND SEISMICITY

1.0 INTRODUCTION

This section presents geologic data on the upper Guadalupe River project area including information obtained during explorations. Included in this section are: (a) a general description of the regional geology; (b) a detailed geologic description of the project area; (c) an assessment of groundwater conditions; (d) a review of land subsidence; (e) a description of subsurface investigations; and (f) regional and site-specific seismicity.

2.0 REGIONAL GEOLOGY

2.1. - General

The discussion of the regional geology will first discuss the general geography of the project area and then discuss in general terms the regional geology.

2.2. - Geography

The project area covers the upper reaches of the Guadalupe River and lies in the highly urbanized Santa Clara Valley in the northern part of the Santa Clara County, California. Santa Clara County is located in the central Coast Range Mountains of California and at the southern end of San Francisco Bay. The Guadalupe River flows north out of the Santa Cruz Mountains, through the Santa Clara Valley, and empties into the southern end of San Francisco Bay. The portion of the project that was investigated by subsurface investigation extends approximately 4.2 miles (6.8 kilometers) upstream from Willow Street, City of San Jose, to the new California State Highway 85 freeway overcrossing of the river just downstream of Blossom Hill Road.

The Santa Clara Valley is an alluvial plain bounded on the west and southwest by the Santa Cruz Mountains and on the east by the Diablo Range of the California Coast Range Mountains. The Santa Cruz Mountains and the Diablo Range converge to form a narrows near the small community of Coyote, California. The Diablo Range extends northerly to San Pablo Bay and the Carquinez Strait of the San Joaquin - Sacramento River while the Santa Cruz Mountains extend northwesterly up the San Francisco Peninsula. The topography of the valley ranges from sea level at Guadalupe River - Alviso Slough and Mud Flats at the southern end of San Francisco Bay to elevations 107, 157, and 180 feet (32.6, 47.8, and 54.9 meters respectively) NGVD (National Geodetic Vertical Datum) at Willow Street, Capitol Expressway, and Highway 85 freeway crossing respectively. The Diablo Range rises above the valley floor to an elevation of 4,213 feet (1284.1 meters) NGVD at Mount Hamilton located 12 miles (19.3 kilometers) east of the project. The Santa Cruz Mountains form a series of northwesterly trending ridges that stair-step up in elevation south of the alluvial plain. The first ridge is the Santa Teresa Hills located approximately 4 miles (6.4 kilometers) south of the Capitol Expressway river crossing. This ridge rises to an elevation of about 200 feet (61 meters) NGVD at its western end and attains it maximum elevation of about 1,150 feet (350.5 meters) NGVD at Coyote Peak at its eastern end. The next ridge to the southwest is Los Capitancillos Ridge located approximately 7 miles (11.3 kilometers) from the Capitol Expressway river crossing. This ridge rises in elevation to approximately 800 feet (243.8 meters) NGVD at its northwestern end to elevation 1,750 feet (533.4 meters) NGVD on Mine Hill in the central part. The southeastern part of the ridge is more dissected but generally has peaks that reach similar elevations as Mine Hill. This ridge is transected by the Guadalupe Creek and Alamitos Creek; the two merge together at the northwestern end of the Santa Teresa Hills to form the Guadalupe River.

Both Guadalupe and Alamitos Creeks collect run off from Los Capitancillos Ridge and from the next ridge to the southwest, the Sierra Azul. Both creeks have been dammed to form reservoirs on the southwestern side of Los Capitancillos Ridge. The reservoirs are the Guadalupe and the Almaden. A tributary to Alamitos Creek called Arroyo Calero drains the northern slope of the southeastern part of Los Capitancillos Ridge as well as the southern side of the Santa Teresa Hills. The Arroyo Calero has also been dammed to form Calero Reservoir near the southeastern end of the Santa Teresa Hills.

The Sierra Azul is the third ridge to the southwest and forms part of the backbone of the Santa Cruz Mountains. This ridge attains higher elevations and is much more rugged; it has peaks that extend above elevation 3,000 feet (914 meters) NGVD. Loma Prieta Peak located approximately 11.4 miles (18.3 kilometers) south of Capitol Expressway river bridge reaches an elevation of 3,791 feet (1155.5 meters) NGVD; Mount Umunhum located approximately 8 miles (13 kilometers) south-southwest reaches an elevation of 3,486 feet (1062.5 meters) NGVD. The northwestern end of the ridge is transected by Los Gatos Creek which merges with the Guadalupe River downstream of the project. Los Gatos Creek has been dammed where it breaches the ridge to form Lexington Reservoir.

2.3. - Regional Geology

The upper Guadalupe River project is located on the upper portion of the alluvial plain formed by coalescing alluvial fans at the southern end of the Santa Clara Valley. The San Francisco Bay as well as the surrounding mountains began to take form approximately 2 million years ago near the end of the Pliocene time. The entire region of the central California Coast Ranges was bodily elevated and much of it folded and locally faulted. The area between folds as well as isolated basins became collection sites for the deposition of eroded sediments. Downwarping due to folding and faulting began to form the Santa Clara Valley basin and the San Francisco Bay trough. The Santa Clara basin began collecting alluvial sediments of sand, gravel, silts and clays from the eroding uplifted and folded areas. Those sediments comprise the Santa Clara Formation. The Santa Clara basin appears to have bedrock contours that extend to well below -2,400 feet (-731 meters) NGVD between the Alviso Mud Flats and San Jose (California Department of Water Resources (DWR), 1967).

During the middle of the Pleistocene time (Pleistocene time began approximately 1.8 million years before present and ended approximately 11 thousand years before present) the central California Coast Ranges "... began to break up into blocks, some of which rose while others subsided" (Howard, Arthur D., 1979). Many of the blocks appear to be tilted; the San Francisco Bay trough was canted eastward. The entire Diablo Range south from San Pablo Bay was bodily uplifted and broken by many cross faults, and the thus separated blocks were lifted up either to the north or south (Howard, Arthur D. 1979). The Santa Clara basin continued to subside; major folds and faults originally formed near the end of the Pliocene were accentuated and new folds were developed in the deposited Santa Clara Formation. Even while the Santa Clara Formation was being folded, sediments of the formation were still being deposited from the older Mesozoic (Jurassic and Cretaceous) and Tertiary age rocks from the surrounding uplands. The Santa Clara basin gradually subsided along several parallel northwest trending faults that are now concealed beneath the valley alluvium. Mid-Pleistocene time saw the end of the deposition of the Santa Clara Formation and the beginning of the accumulation of the late-Pleistocene alluvial fill as erosion and deposition responded to the changing depositional base levels. The base levels changed with the change in sea levels due to the advancing and retreating continental ice sheets of the Great Ice Age. Even so, tectonic subsidence has apparently continued through the present and may be on the order of 0.5 mm (0.0016 foot) per year (Atwater, Hedel, and Helley, 1977). The wide spread late-Quarternary deposits occurred as streams deposited sand and gravel, and thick sequences of fine-grained alluvial sediments. (The Quarternary age began with the beginning of the Pleistocene and continues through the present.)

While the Santa Clara basin was subsiding the Santa Cruz Mountains now forming the basin's northwestern boundary, were uplifted. The topographic expression of the mountains was and is being formed by compression normal to the San Andreas fault. The compression being, in part, the result of a left-stepping restraining bend in the San Andreas fault, and in part due to a small regional compressional component normal to the Pacific-North American plate boundary. The 1989 Loma Prieta earthquake (M=7.1) caused the Santa Cruz Mountains west of the San Andreas fault to be uplifted as much as 0.55 meters (1.8 feet) northerly of the epicenter while as much as 0.1 meters of depression occurred east of the fault in the vicinity of Los Gatos and Vasona Reservoir (Marshall, G.A. et al., 1991). The uplift rate of the Santa Cruz mountains east of the San Andreas fault and forming the northwestern boundary of the Santa Clara basin has been estimated at approximately 0.8 mm per year based on geomorphic evidence and fission track data (Burgman, R. et al., 1993, as cited in Hitchcock, C.S. et al., 1994). According to Hitchcock and others (1994) the northeastern margin of the southern Santa Cruz Mountains is formed by a series of southwest-dipping, northwest-trending reverse faults that are, in part, responsible for the uplift of the mountain range block. Those faults include the Monte Vista, Shannon, and Berrocal faults. In addition, Hitchcock and others (1994) also indicate some of the uplift movement may possibly be contributed by faults that are apparently buried and traverse the Santa Clara Valley northeast of the range front; they cite the Cascade fault as being one of those faults.

That regional compressional crustal shortening is occurring normal to the San Andreas fault is evident from the folding and seismic activity along the eastern side of the Diablo Range of the central Coast Ranges. Folding and faulting is evidenced by the continuing deformation of the

Coalinga anticline and the north, middle, and south domes of the Kettleman Hills (Bloch, R.B. et al., 1993); and by the 1983 Coalinga M=6.5 earthquake and the 1985 Kettleman Hills North Dome M=6.1 earthquake on separate blind thrust faults. As stated by Dr. David L. Jones (1992) of the University of California at Berkeley: "The presence of large-amplitude folds and related thrust faults that deform young rocks throughout the central Coast Ranges, however, shows that an important component of contraction has existed since Pliocene time and continues today." The statement by Dr. Jones was made concerning the tectonics of a transpressive Pacific-North American plate boundary.

The bedrock for much of the area is the extensively folded, faulted, and sheared Franciscan Complex of Jurassic-Cretaceous age. The Franciscan Complex is the basement rock for much of the Coast Range Mountains in northern and central California. Within the central part of Coast Ranges and adjacent to the project area, the Franciscan Complex consists mainly of medium- to fine-grained graywacke sandstone and dark shale; it also contains lesser quantities of altered mafic rocks commonly called greenstones and small amounts of conglomerate, limestone, and chert (Bailey and Everheart, 1967). Another rock type that is generally associated with the Franciscan Complex is serpentinite. The serpentinite has generally been intruded along faults such as in the Santa Teresa Hills and as plugs found in the Los Capitancillos Ridge. Serpentinite also occurs as a mass comprising most of Oak Hill, immediately east of the Almaden Expressway and north of Canoas Creek.

Cretaceous rocks of Great Valley Sequence affinity are found within the Santa Teresa Hills, the western flank of the Diablo Range east of the Hayward-Evergreen fault, and as fault slivers immediately east of the San Andreas fault. Cretaceous rocks are found in fault contact in the Franciscan Complex. These Cretaceous rocks are marine sandstones and shales (Wagner, Bortugno, and McJunkin, 1990). The Upper Cretaceous rocks are represented by the Berryessa Formation, Panoche Formation, and an unnamed Upper Cretaceous formation. Conglomeratic rock and pebbly sandstone interpretated as the Oakland member of the Chico Formation have been found exposed along Coyote Creek north of the Santa Teresa Hills along the eastern side of the Santa Clara Valley (Helley and Wesling, 1990). The Cretaceous sandstone found in the Santa Teresa Hills has a stratigraphic thickness of 1,200 feet (366 meters) and a few fossils similar to those in the Chico Formation (Bailey and Everhart, 1964).

Tertiary marine sandstone identified as the Temblor Formation of Oligocene age (Wagner and others, 1990) occurs in the Santa Teresa Hills where it lies unconformably with a small angular discordance on Upper Cretaceous rocks and in turn is partially overlain by alluvium (Bailey and Everhart, 1964). The Oligocene sedimentary sequence is fissile shale, fossiliferous limestone, and coarse-grained sandstone.

2.4. - Geologic Structure

The dominant structural pattern of the central Coast Ranges is the parallel, northwest trending folds, faults, and structural blocks of the basement complex. The Franciscan Complex was deposited and accreted to the North American Continent in a subduction zone formed by the once existing Farallon oceanic plate in the eastern Pacific Ocean being subducted under the continental plate. The time period for the deposition and subduction was late-Jurassic to middle-Tertiary time. The Franciscan Complex is believed to extend eastward to the Sierra Nevada granitic-metamorphic basement beneath the Central Valley of California. The Franciscan also extends westward to the San Andreas fault zone. The San Andreas fault zone is a strike-slip transform fault along which the Pacific Ocean plate with the Salinian block granitic-metamorphic basement is sliding northwest. Within the Santa Cruz Mountains, part is attached to the North American plate and part is attached to the Pacific Ocean plate. The San Andreas fault zone swings to a more westerly strike as it passes through the Santa Cruz Mountains giving rise to strike-slip fault movement with an oblique reverse component. The fold axis in the Franciscan commonly strikes N40^0 to 60^{0}W. However, within the Santa Cruz Mountains the Santa Teresa Hills and Los Capitancillos Ridge appear to have a more westerly trend which maybe related to the more westerly trend of the San Andreas fault zone and the inferred strike of early- to late-Pleistocene faults concealed beneath the valley alluvium.

2.5. - Mining

Quicksilver. Within the Santa Cruz Mountains bordering the southwestern edge of the Santa Clara Valley and lying within the drainage basin of the Guadalupe River lies an historic mining area, the New Almaden Quicksilver Mining District. The main portion of the district lies on Los Capitancillos Ridge between Alamitos Creek and Guadalupe Creek. Mining and quicksilver (a common name for the element mercury (Hg)) production began in 1846. This mining district became one of the great mercury mines of the world (Bailey and Everhart, 1964). Several of the mines within the district are the New Almaden, American, Providence, Enriquita, San Antonio, San Mateo, Senator, and Guadalupe mines. Since 1890 the production of mercury was small as compared with earlier output. The New Almaden and Guadalupe mines operated intermittently during the periods 1926 to 1972 and 1920 to 1972 respectively. The mines were operated by small lessees intermittently with more continuous production during the late 1930's and during World War II (Bailey and Everhart, 1964). From 1948 to 1968 the New Almaden mine was worked intermittently by lessees. (A lessee is one to a few miners that would lease discrete portions of the mine from the property owner, paying a royalty to the owner on the mercury produced.) By 1968 the New Idria Mining and Chemical Company had acquired control of the mines on Los Capitancillos Ridge. However, by early 1971 the price of mercury had dropped, making mining profitability very questionable. Mining in the New Almaden District had essentially ceased by the end of 1972. Los Capitancillos Ridge is now the Almaden Quicksilver County Park. Mercury from the mining district found its way into Guadalupe Creek and Alamitos Creek. Currently the fish in both the Guadalupe and Almaden Reservoirs are monitored for bioaccumulated levels of mercury.

Some mining of mercury ore, cinnabar, occurred in the Santa Teresa Hills. Two mines, the Santa Teresa and Bernal were developed prior to 1902. The Santa Teresa mine had apparently not been operated for more than 20 years when investigated by Bailey and Everhart (1964), and the Bernal mine was inactive in 1947 with very little prospect for future development (Bailey and Everhart, 1964).

Sand and Gravel. Sand and gravel mining in the Guadalupe River began in the 1930's in the vicinity of Blossom Hill Road and expanded progressively downstream into and downstream of Highway 85 freeway crossing (Woodward-Clyde Consultants, April 1989). The mining ended in the late 1960's to early 1970's when percolation ponds were created in order to recharge groundwater aquifers seriously depleted of water by deep pumping. By 1968 the river channel had been straightened out and the outline of the percolation ponds had been established (Woodward-Clyde Consultants, April 1989). Mining may have also occurred on Guadalupe and Alamitos Creeks near their confluence upstream of Blossom Hill Road, the locations of existing percolation ponds.

Sand and gravel mining also occurred on Los Gatos Creek, a tributary of the Guadalupe River downstream of the project area. Sand and gravel were mined from Los Gatos Creek since before 1926 in the region of and upstream of Vasona Percolation Dam (Bailey and Everhart, 1964). Bailey and Everhart (1964) reported that the dam built in 1936 was located just downstream of the gravel plant. The further most upstream gravel deposit was reported as having been mined since 1947.

3.0 SITE GEOLOGY

The project is located along approximately 5.5 (8.9 kilometers) miles of the Guadalupe River within the city limits of San Jose between Interstate Highway 280 (I-280) bridge crossing at the northern or downstream end and Blossom Hill Road at the southern or upstream end. The site geology focuses primarily on 4.2 miles (6.8 kilometers) of river between Willow Street and Highway 85 freeway bridge crossing just downstream of Blossom Hill Road. The site geology also includes short reaches of both Canoas Creek and Ross Creek, both tributaries to the Guadalupe River that extend upstream from their mouths 1,615 feet (492.2 meters) and 2,710 feet (826.0 meters) respectively. The project reach along Ross Creek was extended farther upstream subsequent to the completion of the geotechnical investigation.

The river has a gentle, slightly sinuous path flowing in a northerly direction with a relatively gentle gradient. River bank elevations range from elevation 107 feet (32.6 meters) NGVD at Willow Street to elevation 180 feet (54.9 meters) NGVD at the Highway 85 freeway bridge crossing. The project also includes approximately 1.0 miles (1.6 kilometers) of the tributary Ross Creek from its mouth to approximately 750 feet upstream of Jarvis Avenue. Both the Guadalupe River, Canoas Creek and Ross Creek have had their channel influenced and altered by man.

The project area is underlain by several hundred feet of Quaternary age alluvial deposits which overlies Jurassic-Cretaceous to Tertiary age bedrock. Bedrock elevation is variable, ranging from deeper than -800 feet (-244 meters) NGVD below Willow Street to serpentinite outcropping of Oak Hill adjacent to the river; it then drops in elevation to approximately -400 feet (-122 meters) NGVD at the Capitol Expressway river bridge, then rising to near elevation 0.0 NGVD at the Blossom Hill Road river bridge (DWR, 1975). The thickness of the Santa Clara formation overlying bedrock is quite variable, having been dissected by erosion prior to being covered by younger Pleistocene and Holocene alluvium.

The Guadalupe River flows northward from the Los Capitancillos Ridge and down the alluvial plain formed by the coalescing alluvial deposits of Alamitos Creek, Guadalupe Creek, which merge to form the Guadalupe River, the Guadalupe River, and Los Gatos Creek. The project area is in the upper portion of the plain where the Guadalupe River had cut into the older, Pleistocene age alluvial fan deposits during the lower sea level stand of the Wisconsin glacial period and then filled in with Holocene age alluvium as the sea level rose to its present height. Alluvial deposition still occurs during flood stages of the previously mentioned streams.

The alluvial deposits of the Santa Clara Valley and those along the Guadalupe River have been mapped and described several different times. The alluvial materials along the river were mapped by Helley and Brabb (1971) as younger, alluvial fan deposits. Between Blossom Hill Road and Willow Glen Way, the alluvium was mapped as young alluvial fan materials that grade headward to terrace and levee deposits of channels incised in older, alluvial fan deposits. Helley and Brabb (1971) described the deposit as consisting of "... moderately well sorted fine sand and silt with minor gravel beds becoming more abundant toward fan heads." From Willow Glen Way to downstream of the I-280 river crossing, they mapped the alluvial materials as fluvial deposits at the outer edge of young alluvial fans, usually in the form of levees between flood basin deposits. The sediments are described as having variable grain size, mainly fine sand, silt, and silty clay.

The description of the alluvial deposits was later modified in Helley and Lajoie (1979). The young, alluvial fan deposit became Holocene, coarse-grained alluvium described as: loose, well drained, moderately sorted, permeable sand and silt, well bedded; forms flat floodplains in narrow valleys, and well drained levees and flat floodplains in broad valleys and on the higher parts of alluvial fans; and gravel becomes dominant toward fan heads. Locally the deposits are said to contain beds of well-sorted silt, sand, and gravel. The outer edge fluvial deposits were renamed Holocene medium-grained alluvium. Helley and Lajoie (1979) described this material as: unconsolidated, moderately drained, moderately permeable fine sand, silt, and clayey silt with occasional thin beds of coarse sand; forms flat floodplains and active levees in broad valleys and at the outer margins of large alluvial fans and or plains. The medium-grained alluvial materials interfinger with and grade into the coarser-grained material along stream levees and on floodplains in broad valleys, and as one progresses up the gradient of the fan.

Wesling and Helley (1989) refined the mapping of the alluvial deposits along the river. Plates 14a and 14b show the geologic surface map of the project area and is essentially taken from Wesling and Helley (1989).

Their map shows a narrow finger of Holocene, alluvial fan deposits extending downstream on the west side of the Guadalupe River from Blossom Hill Road to Bryan Avenue, just downstream from Ross Creek. Downstream of Bryan Avenue, the map shows Holocene, natural, stream levee deposits extending downstream of the I-280 river crossing. On the east side of the river, the map shows Holocene, natural levee deposits extending downstream from Blossom Hill Road to downstream of I-280. The Holocene, alluvial fan deposits are described by Wesling and Helley (1989) as "... brown or tan, medium dense to dense gravelly sand or sandy gravel that grades upward to sandy or silty clay." The alluvial fan surfaces are reported to typically have a slope of about 0.5 to 0.9 percent but may be as high as one percent. The Holocene, natural levee deposits are described by Wesling and Helley (1989) as "loose, moderate- to well-sorted sandy or clayey silt grading to sandy or silty clay." The natural levee deposits are shown to extend out to the west from the river several hundred to a few tens of hundred feet, and are broader in extent on the east side of the river. According to Helley (1990) the thickness of Holocene alluvium including the natural levee deposits ranges from approximately 45 feet at the I-280 river crossing to 47 feet at Willow Street to 35 feet thick at the Western Pacific Railroad crossing just upstream of the Alma Street bridge crossing. One would expect the natural levee deposit to become thinner and coarser in an upstream direction, finally grading laterally and vertically into coarse alluvial fan deposits. Helley (1990) interpreted the top of the Pleistocene alluvium from boring logs using color, texture, penetration resistance, and stratigraphy and topographic position as a basis for his interpretation. He mainly used the location and logs of CALTRANS' borings for State highway bridges.

Subsurface investigation along the Guadalupe River shows that the upper 30 feet of the natural levee deposits consists of interbedded silty clays, sandy clays, silts, clayey sand and silty sand layers of variable thickness. Generally, from Willow Street upstream to Willow Glen Way the upper 30 feet of soil consists predominantly of silty to sandy clays and clayey silts with some clayey and silty sand interbeds. Upstream of Willow Glen Way, the soil becomes increasingly sandier with interbeds of silty to sandy clays and silts. The clays are generally stiff to very stiff, partially drained. Upstream of Bryant Avenue on the west side of the river, the sand content of the soil increased, probably reflecting a gradual change to coarser, natural levee materials and finally to the Holocene, alluvial fan deposit of Wesling and Helley (1989).

Canoas Creek has, for the most part, been channelized downstream of Blossom Hill Road by man. There is very little of the natural stream channel left except perhaps as it skirts the southwestern edge of Oak Hill. Leaving Oak Hill the creek has been channelled across the natural levee deposit of the Guadalupe River. The creek bottom is approximately 11 feet below the ground surface and approximately 15 feet below the man-made levee crest. Subsurface exploration indicates that below the 4-foot ± high levees the soil is generally clayey with a variable sand content. The soil on the south bank appears to have a higher sand content than the north bank. Further upstream along Canoas Creek, but downstream of Blossom Hill Road, the creek channel is in Holocene flood basin deposits (Helley and Wesling, 1990). Helley and Wesling described the Holocene flood basin deposits as "Organic-rich dark clay to very fine silty clay deposits occupying the lowest topographic position either between Holocene levee deposits or Holocene floodplain deposits."

Ross Creek, downstream of Branham Avenue, has been mostly channelized by man. Ross Creek flows northeast from Branham for approximately 0.7 mile to near Hallbrook Drive before turning east-northeast into a relatively straight channel that carries the creek for approximately 1.5 miles to the Guadalupe River. Between Branham and Hallbrook Drive, Ross Creek flows in a narrow finger of Holocene, alluvial fan deposits that have been laid down on older, alluvial fan deposits of Pleistocene age (Wesling and Helley, 1989). This finger of Holocene, alluvial fan deposits may be partly natural levee deposits of coarse material. The channelized section running east-northeast has been excavated in Pleistocene, alluvial fan deposit and in the Holocene, natural levee deposits of the Guadalupe River. Within the project limits, Ross Creek has been channelized in Holocene, natural levee deposits. The creek bottom lies approximately 8 to 13 feet below its banks and 8 to 15 feet below the surrounding ground surface. There are essentially no existing levees along Ross Creek within the project area. Subsurface investigation along Ross Creek indicates that the soil is essentially clayey with variable sand content ranging from little sand to clayey sand; locally some sandy, clayey gravels were also found at or below creek bottom elevation. The south bank appears to have a higher sand content than the north bank; however, the south bank was more extensively investigated due to greater accessibility.

Two faults are believed to exist beneath the project area. One of the two faults is the Santa Clara fault (DWR, 1975). The fault is shown in the DWR (1975) report as extending northwest from the southwestern flank of Tulare Hill at the narrows formed by the Diablo Range and Santa Cruz Mountains, passing the southwest flank of Oak Hill, and extending on towards the southwest edge of the city of Santa Clara. However, the geologic map of the San Francisco-San Jose quadrangle (Wagner and others, 1990) show the fault as branching just northwest of the project. The eastern branch extending under the cities of Santa Clara and Mountain View and finally under the western end of the Dumbarton Bridge at Ravenswood Point while the western branch is shown to extend to the northwest under Los Altos, Stanford University, and Menlo Park. The Santa Clara fault is shown as an early-Quaternary fault buried under the Santa Clara Valley alluvial plain with questionable late-Pleistocene and/or Holocene or later activity. However, the 1975 DWR report indicates that this fault extends vertically to nearly 50 feet of the ground surface, offsetting late-Pleistocene deposits but not Holocene or recent deposits. The other fault is the Coyote Creek-Piercy fault. This fault branches off the Calaveras fault north of the town of Gilroy and passes Tulare Hill on its northeastern flank then turns and follows the southwestern edge of the Edenvale Ridge and then continues west-northwest across Oak Hill (Wagner and others, 1990). The mapped extent ends under the project area in the vicinity of Curtner Avenue. The 1975 DWR report shows this fault as continuing northwest instead of turning along the Edenvale Ridge but being truncated by a north trending fault, north of Oak Hill. This north trending fault is not shown on the map of the San Francisco-San Jose quadrangle (Wagner and others, 1990).

It appears that there is probably still a large amount of uncertainty as to the precise location and alignment of faults buried beneath Holocene to late-Pleistocene alluvium. Earl Pampeyan (1979) indicates in his "Preliminary Map Showing Recency of Faulting in Coastal North-Central California" that fault information in the southern San Francisco Bay-Santa Clara Valley area is not very reliable and that additions and modifications may be necessary throughout the area. The

Coyote Creek-Piercy fault does not even reach Oak Hill according to the map by Helley and Wesling (1990), but turns westward at the southeastern end of Edenvale Ridge and, concealed beneath the Holocene alluvium, would cross under the river in the vicinity of Chard Dr. exit off of the Almaden Expressway. The concealed fault would then continue westward under Los Gatos. If this alignment is correct, the Coyote-Piercy would cross the Santa Clara fault trace. Whether one terminates the other or they both cross each other is not known.

The Coyote Creek-Piercy fault is shown by Bortugno, McJunkin, and Wagner (1991) as having originated during the early-Pleistocene but of questionable activity during late-Pleistocene or later time. They also show the fault as being locally buried or concealed beneath Holocene alluvium and in discontinuous segments. However, the southern end of the fault may be active due to its association with the active Calaveras fault. The Coyote-Piercy fault offsets Plio-Pleistocene deposits but lacks evidence of Holocene displacement (Hart, Smith, and Bryant, 1982).

4.0 GROUNDWATER

The Santa Clara groundwater basin in underlain by essentially non-water-bearing rock and sediments at depths that range from less than 100 feet (30.5 meters) to over 1,500 feet (457 meters). The water bearing formations consists of the Santa Clara Formation and the Pleistocene to Recent Valley alluvium. The groundwater basin can be divided into two basic areas, that area of the basin underlain by confined aquifers and the area underlain by unconfined aquifers (Santa Clara Valley Water District, July 1989). According to the Water District, the area underlain by confined aquifers extends south from San Francisco Bay to approximately the I-280 river crossing and to approximately 0.5 mile (0.8 kilometers) upstream of the Capitol Expressway crossing of Coyote Creek. The area of unconfined aquifers extends from the area of confined aquifers out to the margins of the groundwater basin. The project area lies within the area of unconfined aquifers. The depth to the regional piezometric groundwater level beneath the area of confined aquifers appears to have fluctuated around 80 feet (24.4 meters) since 1970. A deep water well located at Santa Clara Street and Delmas Avenue, San Jose showed depth to water that ranged from approximately 113 feet (34.4 meters) and 119 feet (36.3 meters) in 1972 and 1988 respectively to approximately 42 feet (12.8 meters) and 55 feet (16.8 meters) in 1982 and 1991 respectively according to a hydrograph record furnish by Thomas Iwamura (1992) of the Santa Clara Valley Water District. The piezometric groundwater level is closely monitored by the Water District.

The groundwater level in the unconfined aquifer area within or very near the project area is irregular and subject to perched water conditions. Perched groundwater can result from irrigation of farm land, domestic irrigation, leaky sewer mains, leaky water mains, seasonal rainfall, and high river flows that may provide water to more permeable, discontinuous zones partially isolated by more impermeable materials. Perched groundwater has been encountered in the area from Willow Street to the Western Pacific Railroad crossing upstream of Alma. CALTRANS performed several borings in that area during 1985 for the Guadalupe Freeway project. Several borings were performed in March and in June of 1985. Of the borings performed in March 1985 groundwater levels were measured in four of them. The groundwater elevations ranged from 55.8 feet (17.01 meters) to 97.5 feet (29.72 meters) NGVD, depths of 53.8 feet (16.40 meters) and 11.5 feet (3.51 meters) respectively below the ground surface. Of the several borings performed in June 1985 four of them had water level measurements listed in their logs. The groundwater elevations found in these borings ranged from 71.3 feet (21.73 meters) to 98.4 feet (29.99 meters) NGVD, depths of 45.9 feet (13.99 meters) and 11.6 feet (3.54 meters) respectively below the ground surface. The Guadalupe River channel bottom elevation ranged from approximately 87.5 feet (26.7 meters) NGVD at Willow Street to approximately 96 feet (29.3 meters) NGVD of the railroad crossing.

Groundwater elevations upstream of the Western Pacific Railroad crossing can vary considerably from year to year and probably are dependent on the rate of water withdrawal and rate of recharge. Upstream at the Malone Street bridge crossing, a boring performed in June 1959 encountered groundwater at a depth of 28 feet (8.53 meters). Four borings performed by others in December 1989 encountered groundwater at a depth of 30 feet (9.14 meters), approximately 5.5 feet (1.68 meters) below the channel bottom. Borings performed at the Capitol Expressway river crossing and at the Almaden Expressway-Capitol Expressway crossing for CALTRANS in August 1963 and October 1967 respectively encountered groundwater at different elevations.

The water level elevation in 1963 was 106.3 feet (32.40 meters) NGVD, approximately 32 feet (9.75 meters) below the current channel bottom; and the water level in 1967 was 98.3 feet (29.96 meters) NGVD, approximately 41 feet (12.5 meters) below the current channel bottom. At approximately 2,650 feet (808 meters) upstream of the Capitol Expressway river bridge in a boring performed for the Corps of Engineers in May 1993, the groundwater level was measured at a depth of 25.3 feet (7.71 meters) below the ground surface, water level elevation approximately 140.5 feet (42.82 meters) NGVD, which is approximately 6 feet (1.83 meters) below the river channel bottom. In another boring performed for the Corps of Engineers in May 1993 near the downstream end of Ross Creek, west of the Almaden Expressway, the water level was measured at a depth of 20.5 feet (6.25 meters), water level elevation approximately 148 feet (45.1 meters) NGVD, which is 7 feet (2.13 meters) below the creek bottom and approximately 2 feet (0.61 meters) below the river bottom at the mouth of the creek. The mouth of Ross Creek is approximately 3,900 feet (1189 meters) upstream of the Capitol Expressway river bridge. Woodward-Clyde Consultants (April 1989 and December 1991) found that the groundwater at the Highway 85 Freeway river bridge, very near the upstream end of the project, ranged in elevation from 155 feet (47.2 meters) to 168 feet (51.2 meters) NGVD and varied depending upon the elevation of the water in the percolation ponds.

In general, the Guadalupe River within the project reach appears to be a loosing stream. Groundwater flows away from the river and is recharged by the river as indicated by groundwater elevations below the bottom elevations of the river channel. However, local areas of perched water levels may exist as illustrated by the CALTRANS borings between Willow Street and the Western Pacific Railroad river crossing. Water levels in 3 bore holes were monitored by the Sacramento District, Corps of Engineers, for a project on the lower portion of the Guadalupe River. It was found that for the period of monitoring, June 1987 through 4 January 1988, the water level was essentially below the river bottom except at one location where a perched water level was found at one foot above the river bottom.

5.0 LAND SUBSIDENCE

The Santa Clara Valley has historically experienced significant land subsidence due to over pumping the confined aquifers and the resulting compaction of the confining silt and clay aquitards from the increase of effective stress. The estimated subsidence in central San Jose is 13.4 feet (4.08 meters) from the year 1912 to 1990 as shown on a plot of the hydrograph record of a deep confined aquifer plotted against land subsidence produced by the Santa Clara Valley Water District (Iwamura, 1992). The subsidence was eventually brought under control and essentially stopped in 1968 due to the use of imported water from the State of California Central Valley Water Project brought to the valley through the South Bay Aqueduct. The water from the Central Valley Water Project is used for municipal and industrial purposes, thereby greatly reducing the demand for pumped groundwater. As urbanization has exploded in the Santa Clara Valley following World War II, a large demand was placed on groundwater aquifers for municipal, industrial, and agricultural use. This demand increased despite the reduced use for agriculture as farm land was gobbled up for urban and suburban development. By 1965 the pumping demand for agricultural use had significantly decreased (Poland and Ireland, 1988). With the arrival of water from the Central Valley Project, more water was available for recharging the groundwater aquifers through percolation ponds. The estimated subsidence in the San Jose area from 1967 to 1990 as shown on the subsidence-hydrograph plot from the Santa Clara Valley Water District, as previously mentioned, is approximately 0.6 foot (0.18 meters). This area has a much greater thickness of confining aquitards than in the project area, and is very susceptible to compaction subsidence due to groundwater withdrawal.

The Santa Clara Valley Water District closely monitors the piezometric groundwater levels in the Santa Clara Valley and "... takes an active approach in prudent management of the groundwater in order to avert any longtime decline of piezometric levels within confined areas of the basin" (Iwamura, 1992).

The effect of the groundwater overdraft and resulting subsidence had but a minor impact on most of the project area. The project area lies in the portion of the groundwater basin that is underlain by unconfined aquifers, and is much less susceptible to compaction related subsidence due to groundwater withdrawal. The following tabulation of subsidence was scaled off of two figures that were subsequently photostatically enlarged.

Land Subsidence (approx. ft.)

River Location	1934 - 1967[5]	1960 - 1967[6]
Willow St. Bridge	2.5	1.2
Alma Ave. Bridge	1.5	1.2
Willow Glen Way Bridge	1.0	0.6
Pine St.	0.6	0.4
Curtner Ave. Bridge	0.3	0.15
Lincoln-Almaden Rd.	0.5	0.15
Hillsdale/Capitol Ave.	0.2	0.10
Branham Ave. Bridge	0.15	0.1 - 0.15

It is highly unlikely that subsidence due to groundwater withdrawal will have any bearing on the project. However, tectonic subsidence may have a minor impact. Atwater, Hedel, and Helley (1977) estimate that tectonic subsidence at the southern end of San Francisco Bay may be occurring at a rate of 0.3 to 0.5 mm per year (0.001 to 0.0016 foot per year). Even at a rate of 0.5 mm per year, tectonic subsidence would have a very minor impact during a 100-year project life.

6.0 SUBSURFACE INVESTIGATION

Subsurface investigation consisting of 62 electric cone-penetrometer test probings and 11 borings have been performed along the upper Guadalupe River and its two tributaries, Canoas Creek and Ross Creek. The electric cone-penetrometer test probes were performed in February and March 1992. The borings were performed in May 1993. The logs and location maps of the subsurface explorations can be found in the Geotechnical Report of the Engineers Appendix. The subsurface investigations were concluded prior to nearly doubling the project reach along Ross Creek. No subsequent investigations were performed.

The electric cone-penetrometer test (CPT) probings consists of 54 probings averaging 30 feet (9.14 meters) deep and eight probings that ranged from 45.1 feet (13.75 meters) to 89.1 feet (27.16 meters) in depth. Forty-eight probings, each 30 feet (9.14 meters) deep were placed along approximately 4.2 miles (6.8 kilometers) of the Guadalupe River from Willow Street to near the Highway 85 Freeway river bridge upstream of Branham Avenue. Of the forty-eight CPTs, eight were placed at the bridge crossings of Willow Street, Willow Glen Way, Curtner Avenue, and Hillsdale Avenue. The CPTs at the bridge crossings were the deepest performed averaging 63.9 feet (19.48 meters) in depth. Five CPTs were performed along 0.25 mile (0.4 kilometers) of Canoas Creek between the Almaden Expressway and Nightingale Drive. Nine CPTs were performed along 0.46 mile (0.74 kilometers) of Ross Creek between the Almaden Expressway and Cherry Avenue. The deeper CPTs at the bridge crossings were each planned to be pushed to 100 feet (30.48 meters); however, they met refusal in dense sands and gravels before reaching that depth.

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[5] Poland, J.F. (1971)

[6] Poland and Ireland (1988)

The borings were performed for the purpose of correlating laboratory soil strengths and characteristics with those interpreted by the CPTINT-CPT Cone Interpretation Program, version 4.0, written by Dr. R.G. Campanella (revision dated December 20, 1991). Six of the eleven borings were performed along the Guadalupe River; two borings were performed along Canoas Creek, and three borings were performed along Ross Creek. The borings ranged in depth from 18 feet (5.49 meters) to 40 feet (12.19 meters) and averaged 28 feet (8.63 meters). Two 40-foot (12.19 - meter) borings were drilled, using a hollow-stem auger, for the purpose of measuring depths to free-standing groundwater. One of the 40-foot (12.19 - meter) borings is located along the Guadalupe River and the other near the downstream end of Ross Creek. Two-inch diameter slotted PVC pipe were placed in each hole; the water levels were measured over a 48-hour period before the PVC pipe was removed and the holes backfilled. The groundwater level in each of the two borings was measured immediately after placement of the slotted pipe and then at 24 and 48 hours. The water level in the boring along the Guadalupe River (GR-93-6) had no change at the end of 48 hours when compared to the measurement at the end of 24 hours; the depth to water was 25.34 feet (7.724 meters), only 0.14 feet (0.043 meters) higher than the initial measurement of 24.48 feet (7.462 meters). The boring along Ross Creek (RC-93-1) had an initial depth to water of 20.57 feet (6.270 meters) and a final depth at 48 hours of 20.53 feet (6.258 meters), 0.02 foot (6.1 centimeters) change per 24-hour period. Disturbed and undisturbed samples from the borings were sent to the South Pacific Division Laboratory for testing.

7.0 SEISMICITY

7.1. - General

The Santa Clara Valley lies within an area bounded by the seismically active San Andreas fault system. The San Andreas fault zone lies west of and nearly parallel to the western side of the valley. The fault curves through the Santa Cruz Mountains, cuts diagonally across the San Francisco Peninsula and heads offshore at Mussel Rock in San Francisco County. The San Andreas fault is the transform boundary between the North American continental plate and the northwestward moving Pacific Ocean plate. The fault zone is more than 700 miles (1,125 kilometers) in length, extending from near the California-Mexico border in the south to Cape Mendocino in northern California.

The Hayward and Calaveras-Paicines fault zones lie to the east in and along the western flank of the Diablo Range. The San Andreas fault system displays active right-lateral, strike-slip displacements. Movement along these fault zones can occur very suddenly resulting in an earthquake or can occur very slowly as seismic creep. The greater the horizontal displacement from a seismic event of sudden movement the greater the magnitude of an earthquake with regards to strike-slip faulting. Greater magnitude earthquakes produce stronger ground motion amplitudes and therefore a greater potential to cause damage to man-made structures and to nature. Historically, very damaging to extensively damaging earthquakes have occurred on the fault zones of the San Andreas fault system.

7.2. - Major Active Faults

An active fault exhibits unequivocal geologic evidence for surface displacements during Holocene time and/or instrumentally recorded macroseismic activity positively related to the fault. Plates 15a and 15b show a fault map of the project and surrounding area as taken from Wagner, Bortugno and McJunkin (1990).

San Andreas Fault Zone. The San Andreas fault zone lies approximately 9 miles (14.5 kilometers) west-southwest of the upstream end of project. The Working Group on California Earthquake Probabilities (1990) (here after referred to as the Working Group) has divided that portion of the fault zone that lies within the general San Francisco Bay area into segments. These segments are the southern Santa Cruz Mountains, the northern Santa Cruz Mountains, the San Francisco Peninsula, and the North Coast segments. The fault was segmented for the purpose of assessing the probability of the fault zone for producing large earthquakes, principally those with magnitudes (M) greater than or equal to 7. The end points of the segments occur at points where the strike of the fault zone changes direction. The strike of the fault zone changes from approximately N40^{0}W south of the Santa Cruz Mountains to approximately N52^{0}W in the Southern Santa Cruz Mountains, then to approximately N45^{0}W in the northern Santa Cruz Mountains, to Approximately N37^{0}W on the San Francisco Peninsula, and then to approximately N32^{0}W north of San Francisco. The southern Santa Cruz Mountains is delineated by the rupture zone of the M 7.1 Loma Prieta earthquake of October 1989. The northern segment extends northwest from the Loma Prieta rupture zone to just south of Portola Valley and is approximately 12.5 miles (20 kilometers) in length. The Working Group (1990) assigned this segment an expected M 6.5 earthquake with an 18% probability of occurring within the 30 year period that began 1 January 1990. The San Francisco Peninsula Segment extends northwest from the north end of the Loma Prieta rupture zone (includes the northern Santa Cruz mountains segment) to the San Andreas Reservoir south of San Francisco. This segment was assigned an expected M$\geq$7 earthquake with a 23% probability of occurring within the 30 year period that began 1 January 1990, and a 37% probability of having a M$\geq$6.5 earthquake within the same 30 year period. The North Coast segment extends northwest from the San Andreas Reservoir. This segment was assigned an expected M$\geq$8 earthquake with a 2% probability for the same 30 years period. An earthquake of M 8.3 occurred on this segment in 1906.

Hayward Fault Zone. The Hayward fault zone lies approximately 7 miles (11.3 kilometers) from the project at its closest point. It is approximately 75 miles (120 kilometers) long from San Jose to the northern side of San Pablo Bay and lies along the western flank of the Hayward-Oakland-Berkeley Hills. However, the southern portion of the fault zone, south of the Santa Clara-Alameda County line is not considered to be as active as the rest of the fault zone. Lienkaemper, Borchardt, and Lisowski (1991) regard the southern portion, referred to as the southeastern extension, as being less active and more complex, and having a much weaker geomorphic expression than can be observed to the northwest. Bryant (1982) indicates that while the northern part of the southeastern extension is characterized by a complex zone of geomorphically poorly defined reverse and reverse-oblique-slip faults, further southeast from Alum Rock, east of San Jose, to near Mt. Misery (approximately 7.5 miles or 12 kilometers) the fault zone is characterized by a relatively narrow zone of moderately to well-defined Holocene strike-slip faulting that trends N35°W. Bryant (1982) writes that this segment of the Hayward fault zone appears similar to faulting along the segment that ruptured in 1868; however, there are no observations of fault creep or historic seismicity. Microseismic activity appears to predominately underlie the Calaveras and Mission faults, and the Hayward fault north of Fremont rather than under the southeast extension. However, it should be noted that a M 6.25± earthquake is believed to have occurred at or near the north end of the southeastern extension in November 1858 (Ellsworth, 1990). The Hayward fault, north of the Santa Clara-Alameda County line, has been divided in to two segments by the Working Group (1990), the northern segment and the southern segment. The northern segment extends southeast from the northern edge of San Pablo Bay to San Leandro, California. The northern edge of San Pablo Bay was selected as the north end of this segment because it is the location of an approximate 2,300- to 2,600-foot (700- to 800-meter) wide, 2.5-mile (4-kilometer) long right-stepping bend to the Rodgers Creek fault zone. The southern segment extends southeast from San Leandro to the Santa Clara-Alameda County line. The northern segment was assigned an expected $M \geq 7$ earthquake with a 28% probability of occurring in the 30 year period that began 1 January 1990. The southern segment was also assigned an expected $M \geq 7$ with a 23% probability of occurring during the same 30 year period. Lienkaemper and others (1991) suggest that the October 1868 earthquake, M 7 (Ellsworth, 1990), that ruptured the southern segment had its epicenter in the San Leandro salient of the Hayward fault zone. They suggest that the rupture propagated southeastward and that the Oakland salient acted to stop the propagation to the northwest. They further suggest that the Oakland salient may very well have been the epicenter location for the rupture of the northern segment in June 1836, M 6.75 (Ellsworth, 1990). Accordingly, the 1836 event may very probably have propagated to the northwest with the San Leandro salient having blocked any southeastward propagation.

Calaveras-Paicines Fault Zone. The Calaveras-Paicines fault zone was not included in the probability study performed by the Working Group (1990). The reason it was not included was, there was an insufficient understanding of the segmentation that may be occurring along the fault zone as well as a lack of data on slip rates and displacements for large magnitude events for reaches along the fault (Dr. Robert D. Brown Jr., verbal communication, January 1991). The Calaveras-Paicines fault zone is approximately 79 miles (127 kilometers) long lies approximately 9 mile (14.5 kilometers) from the project at its closest point. The fault zone branches off the San Andreas fault zone southeast of Hollister and trends north along the eastern flank of the Hayward-Oakland-Berkeley Hills. The fault zone passes through Coyote, Anderson, and Calaveras Reservoirs, Sunol Valley, and along the western edge of the Livermore-San Ramon Valleys to just south of Danville. Seismic activity in the Danville area appears to have stepped to the east approximately 3 to 4 miles (5 to 6 kilometers) to the apparent southern end of the Concord fault. The Calaveras Reservoir apparently marks the division point that separates the fault zone into two sections. The seismic activity along the southern section appears to transfer to the Hayward fault zone along the Mission fault which trends from the north end of the Calaveras

Reservoir across the Hayward Hills towards the City of Fremont. This transfer of seismic activity is evidence by the linear plot of epicenter of small magnitude earthquakes (Brown, R.D., 1972). This narrow zone of seismic activity "... defines a thin vertical sheet which coincides with the Mission fault ..." (Ellsworth, Olsen, Shijo, and Marks, 1982, p. 87). However, Ellsworth and other (1982) said that there is an apparent absence of firm geologic evidence for recent movement along the surface trace of the Mission fault.

Approximately fourteen seismic events of $M \geq 5.0$ have been attributed to the Calaveras-Paicines fault zone. Two of those events occurred on the fault north of the Calaveras Reservoir including the earliest known event of M 6.0± in July 1861. South of the reservoir at least 12 event of $M \geq 5$ have been attributed to the fault zone from the year 1866 to June 1988 (Oppenheimer, Bakun, and Lindh, 1990), and none occurring between June 1988 and 30 June 1993. Three of these events had magnitudes believed to be greater than 6.0. The largest event was a M 6.6 in July 1911 (Toppozada and Park, 1982) (Oppenheimer and others, 1990), and the latest of the three was the M 6.2 earthquake of April 1984. Two earthquakes, M 5.3 and 5.4 have occurred along the southern section of the fault since 1984. One of these two events occurred southeast of Hollister on the Paicines fault segment.

According to Oppenheimer and others (1990) they believe that the seismic evidence and isoseismal data indicates that many of the historic earthquakes of $M>5$ are repeat events. They concluded, among other things that:

a. Deformation on the Calaveras fault occurs through aseismic creep above a depth of 5 kilometers (3 miles), combined creep and minor earthquakes occur between 5 and 10 kilometers (3 and 6 miles), and $M>5$ earthquakes occur between 8 and 10 kilometers (5 and 6.2 miles).

b. The analysis of seismic data for six historic earthquakes south of the Calaveras Reservoir, of $M>5$ indicates that three of the six events are repeat events that are related to the other three events.

c. Evidence indicates that aseismic slip after each earthquake $M>5$ is the mechanism that triggers failure on adjacent locked fault segments which are already close to failure.

Thus, it would appear that for the Calaveras-Paicines fault zone, south of the Calaveras Reservoir, the maximum credible earthquake (MCE) magnitude would be equal to the maximum historical earthquake magnitude, M 6.6; its epicenter probably coincident or very nearly so to the April 1984 M 6.2 earthquake. Historically, the MCE assigned to the fault has been M 7.5 by the East Bay Municipal Utility District. Considering the arguments and evidence presented in Oppenheimer and others (1990), this writer is in favor of using a M 6.6 as the MCE.

7.3. - Other Active Faults

Sargent Fault. The Sargent fault is located approximately 9 miles (14.5 kilometers) west-southwest of the project where it branches south eastward from the San Andreas fault zone near Lexington Reservoir. The Sargent fault continues east-southeast to near the Calaveras-Paicines fault zone northwest of Hollister. The fault lies in the apex of the acute angle between the San Andreas and Calaveras-Paicines fault zones. The Sargent fault is an active right-lateral, strike-slip to reverse-oblique-slip fault with a large right-lateral slip component (Hall, Cotton, and Hall, 1980). The fault is approximately 35 miles (56 kilometers) in length. Based upon Slemmons (1977) magnitude versus fault rupture length regression-analysis for combined reverse-slip, reverse-oblique-slip and strike-slip faulting a MCE of M 6.6 is assigned to this fault. It is noted, however, that Wesson and others (1975) assigned a MCE of M 6.9 and Perkins (1987) divided the fault into two segments. North of latitude 37^0N Perkins (1987) assigned it a M 6.4 and south of latitude 37^0N it was assigned a M 5.9. Since the Sargent fault experienced sympathetic seismic activity from the October 1989 M 7.1 Loma Prieta earthquake without triggering any events $M \geq 6$, it would seem that the MCE of M 7.0 of Greensfelder (1974) is too large and that perhaps a M 6.0 to 6.6 may be more reasonable.

Historically the Sargent fault has produced at least five earthquakes $M \geq 5$ but < 6. The earliest reported event that may have occurred on the Sargent fault is the event of February 1864, approximate M 5.9 (Toppozada and Park, 1982). The most recent event is the June 1988 M 5.7 that occurred near its junction with the San Andreas fault zone; although, a M 5.0 event appears to have occurred in October 1989 amongst the aftershocks of the M 7.1 earlier that same month on the San Andreas fault zone. Of the five events that occurred prior to October 1989, all but one occurred on the southern half of the fault. A plot of small magnitude earthquakes for the years 1969 through 1971 (Brown, 1972) shows that the activity on the Sargent fault for that three-year period was predominately along the southern two-thirds of the fault and the majority occurred along the southern one-half, south of the junction of the Berrocal fault with the Sargent fault.

Evergreen-Crosely Fault. The Evergreen fault is a relatively short fault approximately 3.8 miles (6 kilometers) in length that approximately parallels the southern end of the Hayward fault's southeast extension, east of San Jose. It is located approximately 5 miles (8 kilometers) east of the north end of the project. However, Bortugno and others (1991) show the Evergreen combined with the Crosely fault to form an active fault zone approximately 15 miles long (24 kilometers). The fault is shown as a reverse slip fault (Page, 1982). The bases for classifying this fault as active is that Holocene soil and alluvium have probably been offset in three places but no evidence elsewhere although Pleistocene activity is suggested; and the fault is partly well defined (Hart, Smith, and Bryant, 1982). The Crosely fault is listed by Hart and others (1982) as having a scarp that suggest late-Quaternary movement; Holocene soil that has reportedly been offset; and a partly well defined trace. If one assumes that the full length of the fault will rupture during its MCE, that event would be M 6.7 based upon Slemmon's (1977) regression of magnitude versus fault rupture length for combined reverse-slip, reverse-oblique-slip, and strike-slip faults. The MCE could be as high as plus one standard deviation or M 7.3. Perkins (1987) used M 6.9 for the Evergreen fault; the same magnitude was shown for the Hayward fault zone with the Crosely fault included.

The Evergreen-Crosely fault may be a vergent, high angle reverse fault that responds to accumulated contraction strain normal to the Hayward fault and/or the Pacific-North American plate boundary. However, there is insufficient data for selecting a M 7.3 as more likely than other M 6.7 or M 6.9. Even so, for this study a M 6.9 is being used.

Quimby Fault. The Quimby fault is located approximately 6.5 miles (10.5 kilometers) from the northern end of the project and lies east of San Jose. The Trace of this fault sinuously parallels the Hayward fault zone. The fault is relatively short, poorly defined with no evidence of late-Quaternary displacement except where the fault coincides with the Hayward fault zone (Hart and others, 1982).

7.4. - Probably Active Faults

Probably active faults generally exhibit some characteristics of active faults but are not sufficiently well enough defined and do not have sufficiently documented Holocene surface displacement or directly attributable macroseismic and/or microseismic activity. However, these faults exhibit evidence of late-Quaternary activity.

SE Extension Hayward Fault. The southeastern extension of the Hayward fault zone (see discussion on the active Hayward fault zone) is shown by Bortugno and other (1991) as being late-Quarternary but having questionable Holocene activity. Bryant (1982) indicates that part of the fault shows Holocene activity. Hart (1992) shows the full length of the southeastern extension of the Hayward fault zone as being included in the Alquist-Priolo special studies zone for the fault.

Cascade Fault. A fault call the Cascade fault by DWR (1975) is shown unnamed by Bortugno and others (1991) as exhibiting evidence of late-Quaternary activity. The Cascade fault lies approximately 0.7 mile (1 kilometer) south of Blossom Hill Road. The fault is shown by Bortugno and other (1991) as being approximately 23 miles (37 kilometers) in length, extending west-northwest from the north flank of Coyote Peak west of Tulare Hill, along the ridge of the Santa Teresa Hills, then concealed under the Holocene alluvium as it curves to the northwest through Campbell and Cupertino. However, Wesling and Helley (1989) do not show any portion of the fault on their map.

In a study titled "Geomorphic Investigations of Deformation Along the Northeastern Margin of the Santa Cruz Mountains", Hitchcock and others (1994) concluded that the evidence documented in their study strongly suggests that the Cascade fault has undergone late-Pleistocene and possibly Holocene activity. Based on Slemmons (1977) regression analysis of magnitude versus length of fault rupture for combined normal-slip, normal-oblique-slip, and strike-slip faults and assuming a rupture length equal to one-half of the total fault length, the Cascade fault can be assigned a potential MCE of M 6.6.

Shannon Fault. The Shannon fault is located south of and generally paralleling the Cascade fault and lies 2.5 miles (4 kilometers) south of the southern end of the project. This fault begins along the southern flank of Coyote Peak and west of the narrows. It trends along the northern flank of Los Capitancillos Ridge, then curves, and divides into four splays be for passing under Los Gatos Creek between the Camden percolation ponds and the city of Los Gatos. The fault then appears to proceed northwestward with one of the splays possibly joining the southeastern end of the Monte Vista fault. The Shannon fault is shown on the maps of Cooper-Clark (1974, Plate 2-B), Rogers and Williams (1974), and DWR (1967). According to Rogers and Williams (1974, page 16) "the Shannon fault zone is associated with three isolated clusters of small magnitude earthquakes located 1) south of San Jose, 2) near Vasona Reservoir, and 3) West of Cupertino." Near the Vasona reservoir a cluster of M 1-2.5 events appear to be centered on the fault. This fault is approximately 17 miles (27 kilometers) long and appears to branch at its southern end. One of the branches is the Calero fault. An earthquake of M 4.1 occurred in April 1954; its epicenter plotted very close to the surface trace of the fault at the Calero Reservoir south-southeast of San Jose. There has been no unequivocal reported displacement of Holocene deposits. Hart and others (1982) lists the Shannon fault as offsetting Pleistocene sediments locally; it lacks geomorphic evidence of recent faulting; and it is poorly defined.

Bortugno and others (1991) shows the fault as having early Pleistocene activity, but no activity documented as younger than 700,000 years. However, as reported in the general design memorandum of the lower Guadalupe River by the Sacramento Engineering District, U.S. Army Corps of Engineers, Mr. Robert J. Akers, Chief of Geology Branch, California Division of Safety of Dams (11 March 1988, verbal communications) considers the fault to be active and capable of generating a M 6.0 earthquake. The faulting along the Shannon faults has been reported by DWR (1967) to be normal faulting of small displacements. Hitchcock and others (1994) concluded that the evidence documented in their study strongly suggests that the fault has had late-Pleistocene and possibly Holocene activity. The suggested sense of movement being reverse slip on a southwest dipping fault. Tuttle and Sykes (1992) in their re-evaluation of the 8 October 1865 earthquake, which has been suggested by others to have occurred on the San Andreas fault near the epicenter of the October 1989 Loma Prieta earthquake, concluded that their study indicated that the October 1865 event, M 6.5, probably occurred ether in the Santa Clara Valley or eastern foothills of the Santa Cruz Mountains. The Shannon fault, among others, was suggested as a possible source. Based on a regression analysis of magnitude versus fault rupture length for combined normal-slip, normal-oblique-slip, and strike-slip faults (Slemmons, 1977) and an assumed rupture length of one-half of total length, a potential MCE of M 6.4 can be assigned to the fault. However, for this study a MCE of M 6.5 is being used.

Berrocal Fault. The Berrocal fault is a combination of thrust faulting, oblique-slip, and high angle reverse faulting. The fault lies in and along the eastern side of the Santa Cruz Mountains, and, is approximately 4 miles (6.4 kilometers) southwest from the southern end of the project at its closest point. The Berrocal fault extends northwest to west from the approximate mid-point of the Sargent fault to the San Andreas fault in Portola Valley-Woodside area southwest of the Stanford linear accelerator. This fault passes through the foothills of Los Gatos, Saratoga, Cupertino, Los Altos, and Palo Alto. This fault is approximately 37 miles (60 kilometers) long. Bortugno and others (1991) show the fault as having early to mid-Pleistocene activity, but no indication of late-Quaternary or Holocene activity. However, Hart and others (1982) indicate that the northern segment of the Berrocal fault, between Los Gatos and Portola Valley, has geomorphic evidence of late-Quaternary displacements, but that Holocene evidence is lacking; and the fault is locally well defined. In the southern segment, south of Los Gatos, no mention is made as to any potential for late-Quaternary activity.

Hitchcock and others (1994) in their study indicate that most of the deformation found related to the fault occurred along that portion of the fault which trends northwest from the Sargent fault to the Saratoga-Los Gatos embayment of the northeastern front of the Santa Cruz Mountains, stopping before reaching Saratoga. The deformation then appears to step to the northeast to the Monte Vista fault. Accordingly, the length of the fault trace exhibiting evidence suggesting late-Pleistocene and possible Holocene deformation is approximately 25 miles (40 kilometers) in length, 12 miles (19 kilometers) shorter than the overall trace reported above. This same reach of the fault experienced coseismic deformation from the October 1989 Loma Prieta earthquake (Haugerud and Ellen, 1990, as cited in Hitchcock et al., 1994). The Berrocal fault was also considered to be possible source of the October 1865, M 6.5 earthquake (Tuttle and Sykes, 1992).

Wesson and others (1975) assigned a MCE of M 6.7 to the southern segment of the Berrocal fault. The northern segment is called the Black Mountain fault in Wesson and Others (1975) and was assigned a MCE of M 6.7. Perkins (1987) used a M 7.4 for the total fault zone in compiling her intensity maps. If one assumes that only one-half of the total fault length will rupture during a MCE event and uses the regression analysis of Slemmons (1977) magnitude versus rupture length for combined reverse-slip, reverse-oblique-slip, and strike-slip faults, a MCE of M 6.6 can be calculated. The regression analysis for reverse-slip and reverse-oblique-slip faults yields a MCE of M 7.0. However, there is still a large amount of uncertainty concerning the past seismic history of the fault. In this study a MCE of M 6.6 is used.

Monte Vista fault. The Monte Vista fault lies along the northeastern front of the Santa Cruz Mountains, northwest of the Saratoga-Los Gatos embayment of the northeastern front. The late-Pleistocene uplift rate across the Monte Vista fault has been estimated by Hitchcock and others (1994) to be 0.3 ± 0.2 mm per year for a slip rate of 0.4 ± 0.3 mm per year using a 45 degree dip angle. The Monte Vista fault has been interpreted by R.L. McLanghlin (Hitchcock and other, 1994, after R.L. McLanghlin, personal communication, 1993) as being the surface expression of a former blind thrust that has been rotated to a steeper southwest dipping orientation after having breached the axis of its hanging wall anticline. The fault lies approximately 8 miles (13 kilometers) west of the northern end of the project and is approximately 14 miles (23 kilometers) in length. The fault essentially incorporates the northern portion of the previously described Berrocal fault of Hart and others (1982) between the Los Altos Hills and Palo Alto. The southeastern end of the fault extends into the Saratoga-Los Gatos embayment and may join with one of the mapped splays of the Shannon fault. The Monte Vista fault has been estimated to have a potential for a M>6 earthquake (Kovach and Berozo, 1993, as cited in Hitchcock and others, 1994).

For this study it has been assumed that the MCE could rupture the full length of the 14 mile fault. Using the regression analysis of Slemmons (1977) magnitude versus rupture length for a combined reverse-slip and reverse-oblique-slip faults yields a MCE of M 7.0. However, if only one-half of the fault trace ruptured, the regression analysis indicated the resulting earthquake would be M 6.8.

Santa Clara Fault. The Santa Clara fault (previously described under Site Geology) extends from Tulare Hill, passing the south flank of Oak Hill, northward to the western end of the Dumbarton Bridge crossing of southern San Francisco Bay. The fault splits into three branches. The western branch of the fault extends northward to Stanford University and Menlo Park. Wagner and others (1990) show the eastern branch to split into two branches southeast of Mountain View. The western branch of the eastern branch is termed by the writer as the middle branch. The middle branch is believed to extend further north to Foster City, just south of the San Mateo Bridge-Highway 92. However, it should be noted that this fault and its branches are concealed under the valley alluvium and bay mud deposits for the full extent of the fault. There is some question as to the age of the youngest displacement of the fault. Bortugno and others (1991) show the Santa Clara fault as having early to middle-Pleistocene age displacements while DWR (1975) indicates that the latest displacements of the fault could be as young as late-Quaternary south of Santa Clara. For those reasons the Santa Clara fault is placed in the Probable Active category. There is no known macroseismicity that would indicate any historic seismic activity. The fault length ranges from 31.5 miles (approximately 51 kilometers) which includes the western branch to approximately 29 miles (47 kilometers) for the eastern branch plus stem while the middle branch plus stem has a total strand length of approximately 36 miles (58 kilometers). Using the regression analysis of Slemmons (1977) for magnitude versus fault rupture length for combined normal-slip, normal-oblique-slip, and strike-slip faults and assuming a rupture length equal to one-half of the fault length, the Santa Clara could be potentially capable of producing M 6.4 to 6.8 earthquakes. However much is still unknown about this fault and its branches such as age of last displacement, the ratio of rupture length to total length, the amount of displacement during past events, and the magnitude of those events. Should the full length of the fault rupture in one event then the fault could be capable of M 7.1 earthquake. As the fault passes under the project, an earthquake of M 6.4 would produce devastating ground motions that would seriously impact the project features.

7.5. - Potentially Active Faults

Potentially active faults are those that have shown evidence of activity during the early to middle Pleistocene but no conclusive evidence of actively during the late-Pleistocene or Holocene, and little or no historical microseismicity.

Silver Creek Fault. The Silver Creek fault has been postulated to be approximately 31 mile (50 kilometers) in length and is nearly parallel to the Hayward fault zone which lies a few miles to the east. The Silver Creek fault is approximately 2.5 miles (4 kilometers) northeast at its closest point to the north end of the project. Rogers and Williams (1974) show the Silver Creek fault extending northward from near the Calaveras fault east of Morgan Hill. The DWR (1967) report projects the fault beneath the Holocene alluvium and bay mud to 1.5 east of the northern end of the Coyote Hills and just south of Highway 92. The fault was projected to its northern limit by DWR (1967) based on the results of a gravity survey conducted in the performance of evaluating the groundwater resources of the southern San Francisco Bay. Hart and others (1982) listed the Silver

Creek fault as having offset Plio-Pleistocene units but that evidence of Holocene displacements are lacking; the fault is partly well defined.

While the projected portion of the Silver Creek fault northward of San Jose is based on interpretation of gravimetric data, there has not been any significant seismic activity associated with the projected trace; nor is the buried extension known conclusively to have any affect on groundwater as a barrier or to have offset water bearing aquifers. However, the southern portion of the fault, south of San Jose's Tully Road to the Calaveras fault, approximately 13.5 miles (22 kilometers) in length has shown Pleistocene activity and may be capable of sympathetic movement from large events on the Calaveras fault zone. Although, it is probable that most of the tectonic stress and strain are being transferred to the more active, creeping Calaveras fault zone. Wesson and others (1975) assigned a MCE of M 6.2 to the fault; Perkins (1987) used a M 7.1 for use in developing the maximum intensity map. If one considers the southern 13.5 miles (22 kilometers) of this fault as being capable producing earthquakes, the regression analysis of Slemmons (1977) for strike-slip fault with the assumption of a rupture length equal to one-half of the 13.5 miles yield a MCE of M 6.1 to M 6.7 at plus one standard deviation. The regression analysis of Slemmons (1977) for combined normal-slip, normal-oblique-slip, and strike-slip faults with a fault rupture length of one-half fault length yields a MCE of M 6.3 to M 6.9 at plus one standard deviation. Since the Calaveras fault zone is probably absorbing most of the tectonic strain a MCE of M 6.2 maybe reasonable.

Coyote Creek-Piercy Fault. The Coyote Creek-Piercy fault (previously described under Site Geology) is concealed under the Holocene alluvium except where it is exposed in outcropping of Tertiary and older bedrock. This fault is indicated as having early to middle-Pleistocene activity but no late-Quaternary activity (Bortugno and others, 1991). Hart and others (1982) indicate that the youngest unit offset on either the Coyote Creek or the Piercy is Plio-Pleistocene in age (Santa Clara formation) with no evidence of Holocene activity, and the faults are partly well to not well defined geomorphically. The fault length as shown by Wagner and others (1990) is approximately 24 miles (39 kilometers). Using the relationship of magnitude versus fault rupture length developed by Slemmons (1977) for combined normal-slip, normal-oblique-slip, and strike-slip faults with a rupture length of one-half of the fault length, one obtains a calculated MCE of M 6.6 to 7.1 at a plus one standard deviation. Perkins (1987) used M 6.8 for the Coyote Creek fault and M 6.4 for the Piercy fault in the development of her maximum intensity maps. However, as for the Silver Creek fault, the creeping and seismically active Calaveras fault zone appears to be relieving most of the tectonic strain being transferred to it from the San Andreas fault zone.

7.6. - Inactive Faults

Inactive faults are those that show no evidence of movement or seismic activity during the last 1.8 to 2 million years, Quaternary time. There are several inactive faults south of the project in the Santa Cruz Mountains east of the San Andreas fault zone, and many inactive faults east of the Calaveras fault zone in the Diablo Range. Between the San Andreas and Hayward-Calaveras fault zones the inactive faults include the Ben Trovato, Soda Springs, Rincon, Metcalfe, Arroyo

Aguague, Almaden, Enriquita, Limekiln, and Coyote Peak to name a few (Cooper-Clark & Associates, 1974, Plates 2A and 2B).

7.7. - Ground Motion Response

The ground acceleration that can be expected from earthquake activity have been determined using the MCE and/or expected earthquake magnitudes for the known active and probable active faults. The peak accelerations were calculated using the attenuation equations for mean peak acceleration versus distance from the seismic source. The attenuation equations used in this study were developed and published by the following noted experts: E.L. Krinitzsky, Frank K. Chang and Otto W. Nuttli (1987); K. Sadigh, C.-Y. Chang, N.A. Abrahamson, S.J. Chiou, and M.S. Power (1993); I.M. Idriss (1991); D.M. Boore, W.B. Joyner and T.E. Fumal (1993); and K.W. Campbell and Y. Bozorgnia (1994). The ground motions were determined for alluvial sites. Two of the sources for attenuation equations, Sadigh and others (1993) and Idriss (1991), developed equations for accelerations for bedrock sites only, and therefore required a conversion factor in order to obtain the mean peak acceleration for alluvial sites. The conversion factor used was the graphic relationship between bedrock acceleration and acceleration for soil site as provide by Seed, Dickenson, and Mok (1994). The graphic relationship for site class combination B + C_1-C_3 was used as it is thought to represent the soil site conditions of the project. The mean peak accelerations determined for the active seismic sources based upon the maximum expected magnitudes are presented in Table 1. Using the mean peak accelerations for those events included in Table 1, with the exception of the northern segments of the San Andreas and Hayward faults, a design acceleration of 0.30g was selected. It is noted that some of those determined mean peak accelerations are greater than 0.30g as well as practically all of the plus one standard deviations. The design acceleration is the average of the mean peak accelerations excluding those for the northern segments of the San Andreas and Hayward faults.

A design ground acceleration of 0.30g is thought to be a reasonable value for such features of the project as slope stability of a widened channel, cribwalls where required, and for floodwalls. The use of higher values of ground acceleration for design is thought to present unnecessarily large design restrictions. The ground acceleration design value exceeds the 0.20g recommended for concrete structures within seismic zone 4 as previously designated by the Uniform Building Code. However, the design acceleration selected may not be appropriate for use in designing the new or modified bridge crossings required by the project. Local building codes pertaining to highway bridge design should be consulted.

The mean peak horizontal ground acceleration determined for the expected magnitude events on the probably active faults are presented in Table 2. The mean peak accelerations for these events are generally higher than for the active faults. However, the accelerations from the probably active faults were not considered in selecting the design accelerations. The acceleration from an expected magnitude event on the probably active Santa Clara fault was not calculated; it was felt that since it passed under the project it could produce accelerations far in excess of these determined for all the other faults.

The mean peak horizontal ground accelerations for the potentially active faults were not determined for this study. It must be understood that the ground accelerations presented in the preceding paragraphs are only best estimates. Phenomena may occur that could locally cause higher ground accelerations. One of these phenomena is call "directivity focusing" which means that an earthquake's energy may be concentrated in a particular direction rather than emanating equally in all directions. Such focusing generally occurs when the fault rupture propagates predominantly in one direction along the fault. Directivity focusing has been suggested as the reason for a very high peak acceleration of 1.3g at Coyote Dam from the April 1984 M 6.2 earthquake on the Calaveras fault (Shakal, Gay, and Sherburne, 1984), and as suggested for the enhanced ground motions recorded from the August 1979 M 5.9 earthquake on the Calaveras fault (Singh, J.P., 1982). Directivity focusing affects on the project could occur from probable active faults underlying or very close to the project, and possibly, from a MCE on the North Coast segment of the San Andreas fault zone and southern segment of the Hayward fault zone.

TABLE 1
ACTIVE FAULTS
GROUND ACCELERATION MEAN PEAK/MEAN PEAK + 1-SD[*]

FAULT	DISTANCE FROM PROJECT	MAGNI-TUDE	KRINITSZKY ET AL. (1987)	SADIGH ET AL. (1993)[**]	IDRISS (1991)[**]	JOINER ET AL. (1993)	CAMPBELL & BOZORGNIA (1994)
San Andreas (Peninsula Seg.)	16.4 km to Capitol Ave.	Mw 7.0	0.34g/0.60g	0.31g/0.40g	0.31g/0.40g	0.26g/0.44g	0.28g/0.42g
	17.7 km to Willow St.	Mw 7.0	0.32g/0.56g	0.30g/0.40g	0.30g/0.39g	0.24g/0.42g	0.27g/0.39g
San Andreas (No. Santa Cruz Mts. Segment)	16.4 km to Capitol Ave.	Mw 6.5	0.26g/0.44g	0.26g/0.37g	0.26g/0.37g	0.20g/0.34g	0.21g/0.31g
	17.7 km to Willow St.	Mw 6.5	0.24g/0.42g	0.25g/0.34g	0.25g/0.35g	0.19g/0.32g	0.19g/0.28g
San Andreas (No. Coast Seg)	78.0 km to Willow St.	(Ms 8.3) (Mw 8.0)	0.21g/0.39g	0.12g/0.17g	0.19g/0.25g	0.14g/0.23g	0.13g/0.20g
	84.2 km to Capitol Ave	(Ms 8.3) (Mw 8.0)	0.19g/0.36g	0.10g/0.16g	0.17g/0.23g	0.13g/0.22g	0.11g/0.18g
Hayward (No. Segment at Oakland Salient)	51.5 km to Willow St.	Mw 7.0	0.12g/0.22g	0.10g/0.17g	0.12g/0.19g	0.11g/0.19g	0.08g/0.14g
Hayward (So. Segment at Southern end)	17.7 km to Willow St.	Mw 7.0	0.39g/0.68g	0.30g/0.40g	0.30g/0.39g	0.24g/0.42g	0.27g/0.39g
	24.1 km to Hy 85 x-ing	Mw 7.0	0.29g/0.52g	0.24g/0.32g	0.24g/0.33g	0.20g/0.33g	0.20g/0.30g

ACTIVE FAULTS (CONTIN'd)
GROUND ACCELERATION MEAN PEAK/MEAN PEAK + 1-SD*

FAULT	DISTANCE FROM PROJECT	MAGNI-TUDE	KRINITSZKY ET AL. (1987)	SADIGH ET AL. (1993)**	IDRISS (1991)**	JOINER ET AL. (1993)	CAMPBELL & BOZORGNIA (1994)
Calaveras	14.5 km to Willow St.	Mw 6.6	0.33g/0.55g	0.30g/0.40g	0.27g/0.38g	0.23g/0.39g	0.25g/0.36g
Evergreen-Crosely	8.0 km to Project	Mw 6.9	0.65g/1.00g	0.48g/0.64g	0.45g/0.59g	0.44g/0.75g	0.50g/0.74g
Sargent	14.5 km to Hy 85 x-ing	Mw 6.5	0.23g/0.45g	0.30g/0.40g	0.27g/0.38g	0.23g/0.40g	0.28g/0.42g

* The ground accelerations used in this table are horizontal for alluvial soil sites.

** These ground accelerations for alluvial soil sites were determined by applying the calculated rock acceleration from the appropriate attenuation relationship to the graphic relationship provided in Seed, Dickenson, and Mok (1994) "Site Effects on Strong Shaking and Seismic Risk: Recent Developments and Their Impacts on Seismic Design Codes and Practice."

TABLE 2
PROBABLY ACTIVE FAULTS
GROUND ACCELERATION MEAN PEAK/MEAN PEAK + 1-SD*

FAULT	DISTANCE FROM PROJECT	MAGNITUDE	KRINITSZKY ET AL. (1987)	SADIGH ET AL. (1993)**	IDRISS (1991)**	JOINER ET AL. (1993)	CAMPBELL & BOZORGNIA (1994)
Berrocal	5.8 km to Hy 85 x-ing	Mw 6.6	0.57g/1.07g	0.50g/0.70g	0.47g/0.64g	0.42g/0.71g	0.54g/0.80g
	8.8 km to Willow St.	Mw 6.6	0.48g/0.89g	0.43g/0.59g	0.41g/0.56g	0.34g/0.58g	0.44g/0.65g
Cascade	1.6 km to Hy 85 x-ing	Mw 6.6	0.70g/1.31g	0.61g/0.86g	0.54g/0.75g	0.54g/0.92g	0.67g/0.99g
	6.0 km to Willow St.	Mw 6.6	0.57g/1.06g	0.44g/0.60g	0.44g/0.59g	0.41g/0.70g	0.53g/0.79g
Monte Vista	12.9 km	Mw 7.0	0.52g/0.97g	0.40g/0.52g	0.39g/0.50g	0.36g/0.61g	0.39g/0.58g
Shannon	4.3 km to Hy 85 x-ing	Mw 6.5	0.57g/1.07g	0.52g/0.72g	0.46g/0.64g	0.43g/0.73g	0.58g/0.86g
	9.5 km to Willow St.	Mw 6.5	0.41g/0.79g	0.38g/0.52g	0.37g/0.50g	0.31g/0.52g	0.40g/0.60g
Hayward (SE Extension)	11.3 km	Mw 7.0	0.57g/0.92g	0.38g/0.49g	0.36g/0.46g	0.33g/0.56g	0.36g/0.53g

* The ground accelerations used in this table are horizontal for alluvial soil sites.

** These ground accelerations for alluvial soil sites were determined by applying the calculated rock acceleration from the appropriate attenuation relationship to the graphic relationship in Seed, Dickenson, and Mok (1994) "Site Effects on Strong Shaking and Seismic Risk: Recent Developments and Their Impacts on Seismic Design Codes and Practice."

8.0 LIQUEFACTION

The soils encountered in the subsurface exploration borings have been viewed with regards to potential for liquefaction resulting from horizontal ground motions produced by an earthquake. In order for a soil to liquefy it must be saturated; it must essentially be granular and non-cohesive; it must be in a relatively loose state, able to obtain a denser grain-packing state; and it must be subjected to sufficient horizontal acceleration to cause individual grains to move and re-orient themselves. When the above conditions are met, the sudden shear strain imparted from earthquake ground shaking produces a very rapid rise in pore water pressures resulting in a significant loss of soil shear strength. The significant loss of shear strength enables the soil to be easily deformed and flow due to the effects of gravity. Ground deformation as a result of earthquake induced liquefaction can generally be classified as either a deformation failure or a flow failure. Deformation failures include lateral spreading and lurch cracking. Flow failure describes the case of a liquefied soil subjected to continuous, very large shear deformation; the soil mass flows until arrested by a decrease in ground slope or other resistance to flow.

The data gathered from the exploration borings indicate that liquefaction may occur in the upper 10 feet of soil at boring GR-93-4 if subjected to a 0.3g ground acceleration while submerged during bank-full or near bank-full flows. However, during flows where the water surface is 10 feet (3 meters) or more below the top of bank, liquefaction is not anticipated. While the upper 4.5 feet (1.4 meters) of boring GR-93-3 appears to be susceptible, the high tip resistance in adjacent electric-cone-penetrometer test probe GCPT-14 and the apparent large percent (50%) of fines would appear to preclude any potential for liquefaction. The soil in the upper 10 feet (3 meters) of boring GR-93-5 also appears to be potentially liquefiable. However, GCPT-33 probe performed near the boring location indicates that the potentially liquefiable soil may not have sufficient area/extent to cause significant damage. The rest of the borings along the upper Guadalupe River and Ross Creek indicate that the upper 10 feet (3 meters) of soil is either clayey or has a sufficient fine content to preclude liquefaction. The granular soils encountered below a depth of 10 feet (3 meters) in the borings were generally found to be sufficiently dense enough to preclude liquefaction as indicated by standard penetration test below counts. However, the standard penetration test was not performed on all granular layers encountered, nor was sufficient soil classification testing performed on those layers to totally rule out the potential for liquefaction.

The correlation between the soils encountered in the borings and those indicated in adjacent CPT probes is not 100 percent. In some cases there are noticeable differences. Also, there were only limited locations available for performing exploration borings and soil sampling. Thus, it is recommended that a more extensive exploration boring program be instituted for the preparation of a general design memorandum during the next phase of the study, the preconstruction engineering and design (PED) phase.

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GEOTECHNICAL REPORT

UPPER GUADALUPE RIVER FLOOD CONTROL PROJECT
SANTA CLARA COUNTY, CALIFORNIA
GEOTECHNICAL REPORT

This report presents a geotechnical review of the Valley View Plan (50-year level of protection) and a portion of the Bypass Channel Plan (100-year level of protection), as developed by the U.S. Army Corps of Engineers and the Santa Clara Valley Water District (SCVWD), respectively. This geotechnical review was based on the design project drawing plans submitted for review. It should be noted that the geotechnical subsurface investigations were performed early in the feasibility study, and were scoped towards the National Economic Development (NED) plan. Although the investigations were constrained by difficulties in gaining the needed rights-of-entry to private lands, the data gathered are adequate to develop an array of reasonable designs and cost estimates. Having been primarily directed towards the NED plan, the geotechnical review was not able to cover all aspects of the final recommended plan. Now that a recommended plan has been identified, additional geotechnical investigations and design analyses will be performed during the next design phase, the pre-construction engineering and design phase (PED). During PED the finial design of the recommended plan will be completed, including foundation investigations and analyses for those features not fully covered during feasibility geotechnical review. For example, geotechnical investigations will be performed for those reaches of the project that have subsequently been extended in length in the recommended plan, for the floodwall alignments, and for the foundations of the RCB culverts and the inlet weir located in Reaches 7 and 8.

This report contains a description of the subsurface exploration performed and an evaluation of soil data based on the soil investigation and laboratory testing. In addition, engineering analysis was also performed to assess the stability of the proposed cut slopes.

PROJECT DESCRIPTION

The Bypass Channel Plan has been developed and detailed by the Santa Clara Valley Water District. The project consists of constructing a bypass channel and widening the existing channel along the upper Guadalupe River from the Southern Pacific Railroad crossing upstream to Blossom Hill Road. The location of the project is shown on Plate 16. This project lies along the upper half of the Guadalupe River and the lower reaches of Canoas and Ross Creeks. Modification of the existing channel would provide a flow rate that has a 1-percent chance of annual occurrence or a 100-year level of flood protection. The Valley View Plan, as developed by the US Army Corps of Engineers using SCVWD design concepts, was designed for a 50-year level of flood protection. The NED plan has been identified as the 50-year plan. However, subsequent to performing the following analyses for the proposed NED plan, the selected plan was changed from the 50-year plan to the 100-year plan. Although information for both plans is provided in this report, more effort was expended to analyze the Valley View Plan than expended on the Bypass Channel Plan.

Bypass Channel Plan

Design constraints for the Bypass Channel Plan include modification of the existing channel to contain a one-percent flow rate (100-year level of protection), and mitigation measures to preserve the existing riparian habitat and vegetation along the Guadalupe River. A brief description of the proposed improvement project is presented as follows:

Upper Guadalupe River -- The proposed improvement to upper Guadalupe River consists of constructing gabion and composite bypass channels, widened cribwall channels, channel widening, levees or floodwalls, and bridge replacements. The gabion side slopes are planned to be 1 horizontal to 1 vertical (1:1).

Ross Creek -- The proposed improvement to Ross Creek consists of widening existing channel with 1:1 side slopes, and adding a concrete culvert at Jarvis Avenue and the Almaden Expressway.

Canoas Creek – The 100-year level of protection would be the responsibility of the SCVWD. However, both the Valley View Plan and the Bypass Channel Plan include 20-year level of protection for the reach between Nightingale Drive and the Almaden Expressway. The 20-year level flood protection measures consist of low floodwalls and culverts under Nightingale Drive and the Almaden Expressway.

Valley View Plan

The major limitations for the Valley View Plan are the modifications of the existing channel to contain a 50-year level of protection, measures to preserve the existing riparian corridor, and very expensive real estate acquisition for the project land. In addition, only one bank of the creek would be modified which prohibits the complete channelization along the Guadalupe River at the project site.

The improvements of the 50-year plan feature channel widening on upper Guadalupe River and Ross Creek, bridge replacement and modification, and floodwalls designed to contain the 50-year discharge. Channel widening would be limited to one bank to preserve the existing riparian habitat. The designed side slopes along the bank are 1.5:1, 1:1, 0.75:1, and 1:6 (1 horizontal: 6 vertical), and were prescribed by the SCVWD to be in conformance with the LPP.

SUBSURFACE EXPLORATION

Two subsurface exploration programs were performed at the site. The initial program consisted of performing electronic Cone Penetrometer Tests (CPT's) along the portions of the Guadalupe River and lower portions of Canoas Creek and Ross Creek. They were performed between 25 February and 18 March 1992. Subsequently, nine exploratory borings were drilled in 1993 at the locations designated as GR-93-2 through GR-93-7, CC-93-1 and 2, and RC-93-1 through RC-93-3. The project along Canoas Creek was reduced to a 20-year level of protection after it was considered of being dropped from the study and subsequent to the performance of the subsurface investigations. CPT's were used as a fast way of obtaining large amounts of subsurface data at a cost considerably less than for an equal number of borings.

1992 Field Investigation

The scope of the 1992 field investigation consisted of performing on-site CPT's along a portion of the Guadalupe River and the two tributaries. As part of this task, the CPT data obtained in the field was analyzed and interpreted to provide general information about the stratigraphy and the approximate strength parameters of the soils at each test location.

The field investigation included performing CPT's at sixty-one sites along the project area including those CPT's performed along Canoas Creek. The approximate locations of the tests performed for the investigation are shown on the Engineering Drawings, sheets C-1 through C-4, C-9 through C-12, and C-14, at the end of the technical appendices. The interpreted soil classification for several selected CPT's are shown on Plate 17. The CPT's were performed under the supervision of Dames and Moore consultants. The CPT's are numbered GCPT for those along the Guadalupe River, RCPT for those along Ross Creek, CCPT for those along Canoas Creek, and OP for the location of additional optional CPT's that were performed during the field investigations. Note that CPT numbers GCPT-15, CCPT-3 and 4, and RCPT-2 were not performed due to access difficulties.

The CPT equipment consisted of an Hogentogler electronic piezocone with rod extensions, pushed into the ground by means of a hydraulic jacking system. The jacking system had a 20-ton thrust capacity, and was mounted on a 7-ton truck with a screw anchor capable of providing the additional reactions for the 20-ton thrust.

Prior to each test, the voids in the cone were de-aired and the pore pressure chamber saturated by flushing with glycerine using a plastic syringe. Subsequently, the piezocone was assembled with a saturated filter element, which had been kept fully submerged in a jar. The cone was then hydraulically pushed into the ground at a penetration speed of approximately 2 cm/sec (0.79 in/sec). Tip and sleeve friction resistance, inclination, and pore water pressure were measured at depth intervals of 0.05 m (1.97 in) along the entire depth of testing. The readings were transmitted from the piezocone to a data-acquisition system via a cable pre-threaded down the push rods. The information was transferred to an on-board computer where it was stored in digital form and processed to compute the friction ratio, differential pore pressure ratio, and corrected cone tip resistance, for each depth interval. The correction to the tip resistance is generally made to account for the effects of unequal pore pressures acting on the exposed surface behind the cone tip and on the ends of the friction sleeve.

After each test was completed, a plot was prepared showing profiles of friction ratio (Fs_s/Q_c), friction resistance (F_s), measured tip resistance (Q_c), pore pressure (P_w), and differential pore pressure ratio (change in P / Qc). The corresponding plots for all the tests are presented in Attachment A, on page A1 through A62. It should be noted that the values of tip resistance presented in these plots correspond to the measured values as opposed to the corrected tip resistance. Similarly, the profiles of friction ratio shown in the plots are based on the measured tip resistance values. Generally, the correction to tip resistance is only significant in soft saturated clays where high values of excess pore pressure and low cone resistance occurs; such was not the case on this project. Therefore, corrected tip resistance is generally very close to measured field values.

After completion of each CPT, the resulting hole was completely backfilled with cement grout. Additionally, the test sites through pavement were patched with either a concrete plug or asphalt pavement patch.

1993 Exploration

The subsurface investigations were conducted by Geomatrix Consultants, Inc., during the fiscal year 1993. The investigations consisted of drilling and sampling nine geotechnical borings along the banks of the upper Guadalupe River, Canoas Creek, and Ross Creek, tributaries to the river, at the locations shown on the Engineering Drawings at the end of the technical appendices, sheets C-2 through C-4 and C-10 through C-12. Temporary piezometers were installed in two of the borings to measure ground water level.

The project site primarily consists of the upper Guadalupe River between Willow Street and the bridge crossing of State Highway 85 located just downstream of Blossom Hill Road. The project limits include approximately one-half mile of the Ross Creek Tributary, from its mouth near Branham Lane and upstream past Cherry Avenue to approximately 800 feet upstream of Jarvis Avenue. The project has been extended approximately 2,400 farther upstream than it was at the time the subsurface investigations were performed. The project limits along Canoas Creek have been extended approximately 1,410 feet father upstream than it was at the time the subsurface investigations were performed.

The borings were drilled between 25 May and 28 May 1993. Borings GR-93-2 through GR-93-7 were drilled to depths of 22.5 to 40 feet along the banks of the upper Guadalupe River. Borings RC-93-1 through RC-93-3 were drilled to depths of 18 to 40 feet along the southerly bank of Ross Creek. Borings CC-93-1 and CC-93-2 were drilled to depths of 22 feet and 27.5 feet respectively along the southerly bank of Canoas Creek. Access width on the northerly bank of both Ross and Canoas Creeks was to narrow to permit the use of a truck-mounted drill rig.

Two borings, GR-93-6 and RC-93-1, were each drilled to a depth of 40 feet using a truck-mounted hollow stem auger drill rig on May 25, 1993. The 8-inch outside diameter hollow stem augers were equipped with an 8-inch-diameter finger-cutting rock tooth bit. Following completion of the drilling and sampling, the augers were removed and 40 feet of 2-inch-diameter slotted PVC pipe was placed in the borings for measurement of water levels. Using a well sounder, the water levels in the temporary piezometers were measured immediately and approximately 24 and 48 hours following installation.

The 7 remaining borings were drilled to depths ranging from 18 to 30 feet using a truck-mounted, rotary wash drill rig. The drill rods and solid auger were equipped with a 4-7/8 inch rock bit and drag bit. Following completion of drilling and sampling, the drilling fluid from the rotary wash borings was placed in 55-gallon drums and left near the corresponding borings. The borings were then backfilled with bentonite/cement grout, with the exception of Borings GR-93-2 and RC-93-3. These two borings were backfilled with soil cuttings.

Visual soil classifications of the samples obtained in the borings were made in the field in accordance with ASTM Standard Method Designation D2488-84. Geomatrix Consultants boring logs were prepared in the field by examining the drill cuttings. The field logs of the soil samples were used in conjunction with laboratory test data to complete the final logs shown on Plates 18, 19, 20, and 21. Water levels measured in Borings GR-93-6 and RC-93-1 were recorded on their respective logs. Copies of the field logs are presented in Attachment B, pages B1 through B13. Those field logs provide the primary bases for the classification of the soils shown in the final logs.

Three types of soil samplers were used during the field program. Standard Penetration Test (SPT) sampler (1.375-inch I.D., 2-inch O.D.), thin-walled Shelby tube (3-inch I.D., 36-inch length), and Modified California split spoon sampler (2.5 inch I.D., 30-inch O.D.). The soil samples recovered from the exploratory borings and samples of drilling fluid collected after the borings were completed were delivered to South Pacific Division Laboratory in Sausalito, California on 4 June 1993.

SUMMARY OF SUBSURFACE SOIL CONDITIONS

Subsurface conditions along the project site were evaluated based on the results of the field exploration programs and the available logs of borings performed on the project site. Based on interpolated cross-sections of soil conditions through the proposed construction areas, the following is a summary of the subsurface soils encountered in each of the three areas that were investigated along the project area.

Area A. The field exploration program in Area A consisted of boring GR-93-2 and CPT numbers GCPT-1 through GCPT-11, and OP-1 and OP-2. The subsurface soils encountered at these locations consist predominantly of lean clays and clayey silts in the upper 30 feet. The consistency of this material is generally soft in the upper 8 feet and medium stiff from 8 to 30 feet. Underlying this material are interbedded layers of sand with gravel, silty clays, and sandy silts to the maximum explored depth of 51 feet in GCPT-4.

Area B. The field exploration program in Area B consisted of borings GR-93-3 and GR-93-4 and CPT numbers GCPT-12 through GCPT-14, GCPT-16 through GCPT-22. The subsurface soils encountered at these locations generally consist of a layer of loose to medium dense silty sand to sandy silt that extends to a depth of 17 feet. Below the upper layer, clay is encountered to a depth of about 30 feet. Below 30 feet to the maximum explored depths of 84 feet in GCPT-21, the soil profile consists of interbedded sandy silts, silty sands, and sands.

Area C. The field exploration program in Area C consisted of borings GR-93-5 through GR-93-7, and CPT numbers GCPT-23 through GCPT-40. The subsurface materials are heterogeneous, ranging from lean and fat clays to silty sand, to poorly graded clayey gravel. In general, the logs of the boring and friction cone penetrometer probings indicate that the upper 30 feet of soil consists of interbedded silty clays, silts and variable thickness of silty sand. From Willow Street to Willow Glen Way, the upper 30 feet of soil predominately consists of silty clays and clayey silts interbedded with silty sand. Upstream of Willow Glen Way, the soil increasingly becomes sandier, with interbeds of silts and silty clays.

LABORATORY TESTING

Laboratory testing included some soil classification, direct shear, unconfined compression, and unconsolidated undrained compression tests. All laboratory testing was performed in accordance with the applicable ASTM standards. The results of the laboratory testing are shown in Attachment 6, pages C1 through C83. The final boring logs shown on Plates 18 through 21 provide the soil classifications of the material encountered during subsurface explorations.

Classification Testing

Atterberg limits, grain size analyses, natural water contents, specific gravity, and unit weight tests were performed on some of SPT and Shelby tube samples of the natural soils obtained during the exploration program. In general field classifications were relied on and supplemented by limited laboratory classifications. In this way the laboratory testing was able to stay within the feasibility budgeted cost.

Strength Testing

Three direct shear tests, fourteen unconfined compression (UC) tests, and twenty unconsolidated undrained triaxial compression (UUC) tests were performed on SPT and Shelby tube samples obtained from the project site. The unconsolidated undrained triaxial compression test samples were back saturated prior to the deviator stress application. The results of strength testing indicate that for drained soil samples the peak friction angles are between 19 degrees and 28 degrees.

The UC and UUC test results were reviewed and shear strength data were correlated with the plasticity indexes of the soil samples. Based on the above correlation and direct shear strength test results, the design strengths were selected for the analyses. No correlation was found between undrained shear strengths determined through CPT testing and those determined by laboratory testing. The CPT undrained shear strengths in all comparisons significantly exceeds the laboratory strengths.

SLOPE STABILITY ANALYSES

Three cross-sections along the project alignment were developed for slope stability analyses. The cross-sections analyzed are at Stations 854+00 and 936+00 along Guadalupe River, and Station 9+00 on Ross Creek. These sections were developed based on subsurface conditions encountered in the exploratory borings. Section configurations were estimated based on the preliminary channel modifications dated 12 October 1995. The subsurface conditions in these sections were generalized so that identified subsurface or modification features, which would have an effect on stability, would be included and their influence on stability assessed.

A review of the final channel configurations is highly recommended to be done in pre-construction engineering and design (PED) to confirm that the slopes and elevations presented on the figures, which were obtained from the planning drawings, accurately represent the cross sections analyzed.

Conditions Analyzed

The following conditions were considered for analysis: water level near channel capacity flood event, water level at bottom channel, rapid drawdown, and earthquake loading. These conditions were selected based on channel configuration and construction conditions, and represent the standard analyses discussed in the Engineering Manual EM 1110-2-1913, Engineering and Design, Design and Construction of Levees, US Army Corps of Engineers.

The following is a brief description of these conditions, the assumptions used in the analyses and the criteria selected for each case.

Water level near maximum channel capacity -- This condition is defined as when the water remains near a full flood stage long enough so that the embankment becomes fully saturated and a condition of steady state seepage occurs. Drained shear strengths were used. It should be noted that steady state seepage conditions would be developed, since the channel bank material consists of clayey material interbedded with permeable sands and gravels; development of excess pore water pressure would not occur within the slope. The Corps of Engineer manual recommends a minimum factor of safety of 1.4 for this condition. The stability of the channel embankment was also analyzed for water level at zero feet above the channel bottom.

Rapid Drawdown -- This case represents the condition whereby a prolonged flood stage saturates at least the major part of the embankment and then falls faster than the water can drain. Design shear strengths of soils were based on the drained values. Based on permeable channel bank material, it is anticipated that excess pore water pressure would not be developed in the rapid drawdown loading condition. The Corps of Engineers manual recommends a minimum factor of safety of 1.0 for this condition.

Earthquake -- Earthquake loadings during a bank full condition were not considered when analyzing the stability of channel banks because of the low probability of an earthquake coinciding with periods of high water. However, the loading condition for water level at zero feet above the channel bottom was analyzed. The seismic coefficient for the slope at the project site was assessed to be about 0.3g. The Corps of Engineers manual recommends a minimum factor of safety of 1.0 for this condition.

Method of Analysis

The slopes were analyzed by conventional limit equilibrium methods to evaluate factors of safety against sliding. The slopes were computer-analyzed for trial, circular arc failure surfaces using the UTEXAS3 slope stability analysis computer program. This program performs an automatic search for the circular failure surface having the minimum factor of safety based on various procedures. For this study, Bishop's simplified procedure was selected.

Bishop's procedure incorporates, as basic input data, the geometry of the channel slope, internal zones, the unit weight and shear strength properties of the channel slope and foundation materials. After a trial failure surface has been assumed, the soil mass above the sliding surface is divided into a series of vertical slices. Forces acting on each slice include the earth pressures on its sides, water pressures on its sides and bottom, stresses with associated friction acting on the assumed slide surface, and cohesion along the slide surface. Various trial failures are analyzed until a minimum factor of safety is obtained for the case being studied.

Results of Stability Analysis – Excavated Slopes

Slope stability analyses performed on the proposed cut slopes resulted in unacceptable low factors of safety. The factors of safety are below Corps of Engineers standards. For instance, the soil profile at Station 936+00 was analyzed for a 1 horizontal to 1 vertical excavated slope and for a 1.5 horizontal to 1 vertical excavated slope. For the case where the water level is located at the bottom of the channel, the 1 horizontal to 1 vertical slope was found to have a static factor of safety less than the minimum acceptable value; and the 1.5 horizontal to 1 vertical slope was found to have a static factor of safety of 1.32 as compared to the recommended minimum factor of safety of 1.4. Both of the slopes had a factor of safety significantly less than 1.0 under a seismic coefficient of 0.30g. Further analyses revealed that to meet a factor of safety of 1.00 or greater under a seismic loading of 0.30g a slope equal to or flatter than 3.5 horizontal to 1 vertical would have to be excavated. The analyzed profile consists of three soil layers. The first or upper layer consists of approximately 4.5 feet thick or loose, cohesionless sandy silt and silt with an angle of internal friction of 28.6 degrees. The underlying second layer is 15 feet of cohesive clayey material with an undrained cohesion intercept of 600 pounds per square foot, angle of internal friction assumed to be 0.0 degrees due to an undrained condition. The third layer which underlies the second layer is cohesionless silty sand to sand with an angle of internal friction of 30 degrees. The moist unit weight of all three layers is assumed to be 123 pounds per cubic feet. It was determined that the import of selected granular fill with geogrid reinforcement would increase the factor of safety to meet Corps of Engineers standards.

REMEDIAL DESIGN AND STABILITY ANALYSES

Figures 1 through 3 illustrate typical cross sections in the remedial design along the alignment of the project site. Key features of the design include:

- Remove the native soil within the slope as shown.
- Import and compact free-draining granular fill to provide acceptable factors of safety against shear failure of the slopes.
- Incorporate internal, tensile reinforcements as required over the cut slopes along the channel during construction. Typical sections showing the material and structural components of the reinforced slopes are shown on Figures 2 and 3. Tensile elements, such a geogrids, are placed in layers in 5-foot interval in the granular fill to create a stable, unitary mass.
- Provide internal drainage of the reinforced fill by the use of free draining material such as crushed drain rock.

Figure 1 shows the details of the typical slopes for the proposed criblock retaining wall. Figures 2 and 3 show the typical sections for the proposed stepped gabion and 1:1 cut slopes, respectively.

Stability Analyses of Compacted Fill Slopes

Analyses were performed to model several conditions: water level near maximum channel capacity, water level of a 10 year flood event, the condition during rapid drawdown from maximum flood water level, and long term steady seepage under static and seismic conditions. The water level of a 10-year flood event was selected for slope stability analyses for the Bypass Channel Plan as that event is considered being a common flood occurrence. However, this water level was not considered in the analyses of the Valley View, 50-year Plan. Greater emphasis is being placed on the Bypass Channel Plan than the 50-year plan. The analyses were completed using a computerized version of the Bishop Method employed by the computer program UTEXAS3. The results of the design analysis are presented on Figures 4 through 6.

A friction angle of 35 degrees was selected for the compacted, free-draining granular material. Reinforcement elements were spaced 5-foot on centers and were 30 to 35 feet in length for the Guadalupe River and 20 to 25 feet in length for Ross Creek. Long-term allowable design strength of the reinforcement is shown on Figures 5 and 6. Earthquake loading was represented by applying a horizontal acceleration equivalent to 0.3 times the acceleration due to gravity (g) to the embankment mass.

Results of Stability Analyses – Channel Bypass Plan

Slope Stability -- Station 854+00

Loading Condition -- Water Level at 100-Year Flood Event

As indicated on the project plans, the estimated water level of the 100-year event is located approximately at Elevation 140 feet or about 19 feet above the bottom of the channel. This water level is about 3 feet below the top of a proposed crib wall. The crib wall is designed essentially according to the State of California, Department of Transportation construction details for reinforced concrete crib walls (July 1, 1992) with a vertical batter of 1 horizontal to 6 vertical. To simplify the analysis of the crib wall a water surface of 141.o feet was used as that is the expected maximum water height to be analyzed for in the Valley View Plan (50-year event). The analysis of this condition assumes a steady state seepage condition has been reached in the channel bottom. The results of the stability analysis were summarized in Figure 4. The factor of safety is 1.37 for this condition, which is slightly less than the Corps of Engineers standards of 1.4. The factor of safety can be increased by using a more densely compacted, coarse angular material or, by using tensile reinforcement elements in the fill attached to the crib wall. More detailed site-specific design will be require during PED.

Loading Condition -- Water Level at 10-Year Flood Event

The 10-year flood event water level was not analyzed for as the factor of safety is expected to fall between that of the flood level, discussed above, and the water level at zero depth, discussed below. The factors of safety using Bishop's procedure are very close.

Loading Condition -- Water Level at Zero Feet

In the zero depth water level condition, the water level is placed at the bottom of the channel. The minimum factor of safety for this condition is 1.38, which is minimally below Corps of Engineers standards of 1.4. As previously discussed, using a more densely compacted angular material may increase the factor of safety sufficiently to meet the minimum criteria. More detailed design analyses during PED will be required.

Rapid Drawdown -- Analysis of rapid drawdown conditions indicates a factor of safety of 1.37, which meets Corps of Engineers standards. The critical failure surface is shown on Figure 4. The back material was considered to be completely free draining material; therefore, no excess pore pressures would be generated. It was assumed the fill would have the ability to drain water at the same rate or faster than the river stage would drop.

Earthquake -- The seismic coefficient for the slope at the project site is assessed to be 0.3g. The factor of safety for the zero water level condition is 1.00, which is the very minimum acceptable.

Slope Stability -- Section 936+00

Water Loading Condition -- The results of the stability analysis for the three water levels are presented in Figure 5. For water levels at Elevations 160.5, 155 and 144.5 feet corresponding to 16, 10.5 and 0 feet from the channel bottom, the factor of safety is 1.9, 1.8 and 2.0, respectively. These values meet Corps of Engineers standards.

Rapid Drawdown -- Figure 5 presents the analysis for rapid drawdown. The phreatic surface is shown on the figure. The phreatic surface used in the analysis is conservative as it indicates a non-free draining condition. The indicated factor of safety of 1.3 meets Corps of Engineers standards.

Earthquake -- The seismic coefficient for the slope at the project site is estimated to be 0.3g. For this seismic parameter, the factor of safety for water depths at 10.5 and 0 feet is 1.08 and 1.02, respectively. Higher water surfaces would give higher factors of safety to earthquake loading due to the mass of the water acting on the slope.

Slope Stability -- Section 9+00, Ross Creek

Water Loading Condition -- The results of the stability analysis for the three water levels are presented in Figure 6. For water levels at Elevations 164.5, 160.8 and 153.5 feet corresponding to 11, 7.3 and 0 feet from the channel bottom, the factor of safety is 5.13, 3.31, and 3.09, respectively. These values meet Corps of Engineers standards.

Rapid Drawdown -- Figure 6 presents the analysis for rapid drawdown. The indicated factor of safety is 1.23, which meets Corps of Engineers standards. The phreatic surface analyzed assumed a very conservative condition of full water retention; the surface conformed to the 1 horizontal to 1 vertical, construction slope. Due to the apparent ultraconservatism, the rapid drawdown condition will be reanalyzed during PED.

Earthquake -- The seismic coefficient for the slope at the project site is assessed to be 0.3g. For this seismic parameter, the factor of safety for water levels at Elevation 160.8 and 153 feet is 1.07 and 1.33, respectively.

Results of Reinforced Slope Stability – Valley View Plan

Slope Stability -- Station 854+00

Loading Condition -- Water Level at 50-Year Flood Event

As indicated on the project plans, the estimated water level for the 50-year flood event is located approximately at Elevation 141 feet or about 20 feet above the bottom of the channel. This water level is about 4 feet below the top of proposed cribwall. The analysis of this condition assumes a steady state seepage condition has been reached in the channel bottom. The results of the stability analysis were summarized in Figure 7. The factor of safety is 1.37 for this condition, which is slightly below Corps of Engineers standards. As discussed for the Bypass Channel, it is expected that a more densely compacted coarse angular fill will raise the factor of safety.

Loading Condition -- Water Level at Zero Feet

In this condition, the water level is placed at the bottom of the channel. The minimum factor of safety for this condition is 1.38, which is minimally below Corps of Engineers standards. Again, more detailed design analyses during PED should easily raise the factor of safety.

Rapid Drawdown -- Analysis for rapid drawdown indicates factor of safety of 1.37, which meets Corps of Engineers standards. The critical failure surface is shown on Figure 7. The compacted fill was considered to be completely free draining.

Earthquake -- The seismic coefficient for the slope at the project site is assessed to be 0.3g. The factor of safety for the zero water level condition is 1.00, which is the very minimum acceptable.

Slope Stability -- Section 936+00

Water Loading Condition -- The results of the stability analysis for the three water levels are presented in Figure 8. For water levels at Elevations 168.2, 155 and 147 feet corresponding to 23.7, 10.5 and 2.5 feet from the channel bottom, the factor of safety is 2.82, 2.24, and 2.01, respectively. These values meet Corps of Engineers standards.

Rapid Drawdown -- Figure 8 presents the analysis for rapid drawdown. The phreatic surface is shown on the figure. The phreatic surface used in the analysis is conservative as it indicates a non-free draining condition. The indicated factor of safety is 1.37, which meets Corps of Engineers standards.

Earthquake -- The seismic coefficient for the slope at the project site is estimated to be 0.3g. For this seismic parameter, the factor of safety for a water depth of 2.5 feet is 1.12. This factor of safety meets Corps of Engineers standards.

Slope Stability -- Section 9+00, Ross Creek

Water Loading Condition -- The results of the stability analysis for the three water levels are presented in Figure 9. For water levels at elevations 164.5, 160.8 and 153.5 feet corresponding to 11, 7.3 and 0 feet above the channel bottom, the factor of safety is 5.13, 3.31, and 3.09, respectively. These values meet Corps of Engineers standards.

Rapid Drawdown -- Figure 9 presents the analysis for rapid drawdown. The indicated factor of safety is 1.2, which meets Corps of Engineers standards. The phreatic surface was very conservatively assumed to conform to the constructed 1 horizontal to 1 vertical constructed slope below the high water elevation of 164.5 feet.

Earthquake -- The seismic coefficient for the slope at the project site is assessed to be 0.3g. For this seismic parameter, the factor of safety for water levels at elevation 160.8

feet and 153 feet is 1.07 and 1.33, respectively. These factors of safety meet Corps of Engineers standards.

FLOODWALLS

Valley View Plan

Floodwalls are proposed for Canoas Creek, Ross Creek, and the Guadalupe River. The floodwalls for both Canoas Creek and Ross Creek are similar in height. They are to hold back 4 feet of floodwater and have 1 foot of freeboard. Floodwalls for the Guadalupe River are planned for Reach 7 and Reach 8. The floodwall in Reach 7 is expected to range from 2 feet to 4 feet in height along the east bank of the river, between Alma Avenue and the Western Pacific Rail Road crossing. Geotechnical data near the bank in the location of this wall is sparse, consisting of one CPT probing. The floodwall in Reach 8 is expected to range from 1 foot to 3 feet in height on both banks of the river upstream of the Western Pacific Rail Road crossing to just downstream of Willow Glen Way. The geotechnical data in the general location of the proposed east bank wall consists of two CPT probings and one boring. No geotechnical data was gathered in the location of the proposed wall on the west bank of the river.

Bypass Channel Plan

Floodwalls are proposed for Canoas Creek and will have the same height as the proposed wall for the Valley View Plan. No floodwalls are proposed for Ross Creek. Floodwalls are proposed for the Guadalupe River in Reach 7, Reach 10b, and Reach 12. The floodwall in Reach 7 is expected to similar as for the Valley View Plan and range from 2 feet to 4 feet in height. The floodwall in Reach 10b is expected to be 4 feet in height along the west bank of the river between the crossing of the Almaden Expressway South and the crossing of the Almaden Expressway North. No geotechnical data was gathered in the location of the proposed floodwall. The floodwall in Reach 12 is proposed for the east (right) bank of the river upstream of Branham Lane for a distance of approximately 900 feet. The floodwall is expected to range from 2 feet to 4 feet in height. Geotechnical data gathered in the location of the proposed wall consist of one CPT probing on the left bank opposite the floodwall and another on the left (west) bank farther upstream. The aforementioned data indicates that the soil on the bank opposite from the proposed floodwall ranges from clayey silt to silty sand. The nature of the soils underlying the proposed floodwall will be ascertained during PED.

Floodwall Analyses

A generic design concept of an 8-inch thick, cantilevered, reinforced concrete, "T"-type floodwall is shown in Figure 10. However, no site-specific stability analyses or seepage analysis has been performed. Site-specific analyses will be performed during PED. The design effort during PED will not be limited to a cantilevered "T"-type wall, but will consider other wall types that may be more appropriate to site-specific soil conditions.

A preliminary analysis was performed for the generic five-foot high wall, 4 feet of water plus 1 foot of freeboard. The indicated factor of safety against overturning with water at the top of the wall, uplift pressures considered, is too low. The factor of safety manually calculated is 1.0 while the factor of safety calculated using the Corps of Engineers computer program CTWALL is 1.3. The desired factor of safety should exceed 1.5. Thus, the base dimensions of the "T" wall should be resized. Preliminary analyses of the stability of the wall against sliding and against overturning have not be included with this report since they are not site-specific, thus would serve no purpose.

CRIB WALLS

In addition to the previously discussed slope stability analyses, the crib wall was also analyzed for sliding and overturning. The analyses was performed using the Corps of Engineers CTWALL computer program. For the analyses the water depth was in the channel was assumed to be essentially zero feet, or approximately elevation 120.9 feet NGVD while the water level within the compacted granular fill was assumed to be 2 feet higher. The computer program took into consideration the uplift pressures on the structure when performing the analyses. The calculated factor of safety against sliding is 2.26 while the factor of safety against overturning is 3.56 with 100% of the base in compression. Attachment D contains the analyses for sliding and overturning. The factors of safety against sliding and overturning are expected to remain high as more detailed design raises the slope stability factors of safety.

SHORT TERM INFRASTRUCTURE AND REAL ESTATE IMPACTS

The use of compacted, free-draining granular fill reinforced with geogrids will permit the construction of the desired steep slopes indicated in the Valley View Plan and the Bypass Channel Plan. However, significant short-term impacts will occur. The impacts will be the result of excavating a marginally stable 1.5 horizontal to 1 vertical slope against which the geogrid-reinforced, compacted granular fill can be constructed. Additionally, the maximum steepness of an unsupported slope allowed by the U.S. Department of Labor, Occupational Safety and Health Administration for the soil conditions at the site is 1.5 horizontal to 1 vertical for heights up to 20 feet without benching (Excavations, OSHA 2226, 1995 (Revised)). The results of excavation will significantly affect local infrastructure as well as private real estate. For instance, the removal and reconstruction of parts of Almaden Road and buried utilities would be necessary. Real estate along Ross Creek would be impacted where excavation will extend beyond the width of the existing right away. Geotechnical investigations to be performed in PED will provide additional shear strength data, and hopefully will result in reducing the magnitude of the impacts. However, it is very likely that there will still be some impacts.

RECOMMENDATIONS

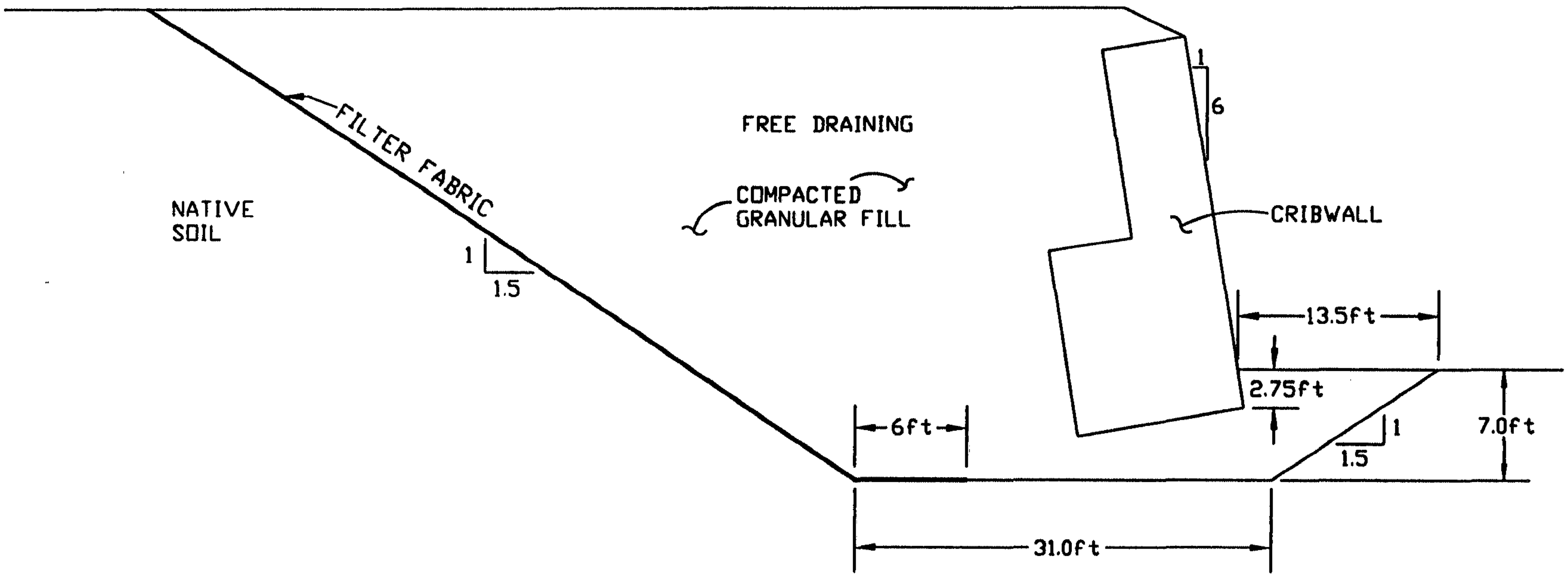
It is recommended that additional borings and laboratory soil strength testing be performed during PED along the Upper Guadalupe River and Ross Creek. The additional exploration is recommended due to the lack of shear strength correlation between laboratory test results and CPT test results, and due to the limited access to the banks of the Upper Guadalupe River available at the time the exploration program was carried out. Future exploration and lab testing results may either confirm existing data and analyses or may show that the selected design strengths based on current test data are too conservative. If current design strengths are found to be too conservative, slopes steeper than 1.5 horizontal to 1 vertical may be able to be safely excavated and used in the short term. However, the steepness of short-term excavation slopes will also have to conform to OSHA's requirements base upon the site-specific soils to be excavated.

A potential alternative to geogrid reinforced compacted fill that should be investigated during PED is the use of a mechanically mixed in place wall to reinforce the streambank. A mixed in place wall would stabilize the bank and allow for the excavation of a near vertical slope; thereby precluding the extensive excavation needed to construct a geogrid-reinforced slope. The near vertical slope could then be modified to a 1 horizontal to 1 vertical by the use of gabions. A mechanically mixed in place wall is constructed by the introduction of cementitious grouts down through the stems of large diameter flight augers. The augers may range from 22 to 40 inches in diameter and are discontinuous. A small amount of grout is placed in the boring and mixed with the disturbed soil in the borehole during the withdrawal of the auger(s), thus forming soil-cement columns. With proper sequencing and spacing of the columns, a continuous in sites wall can be constructed.

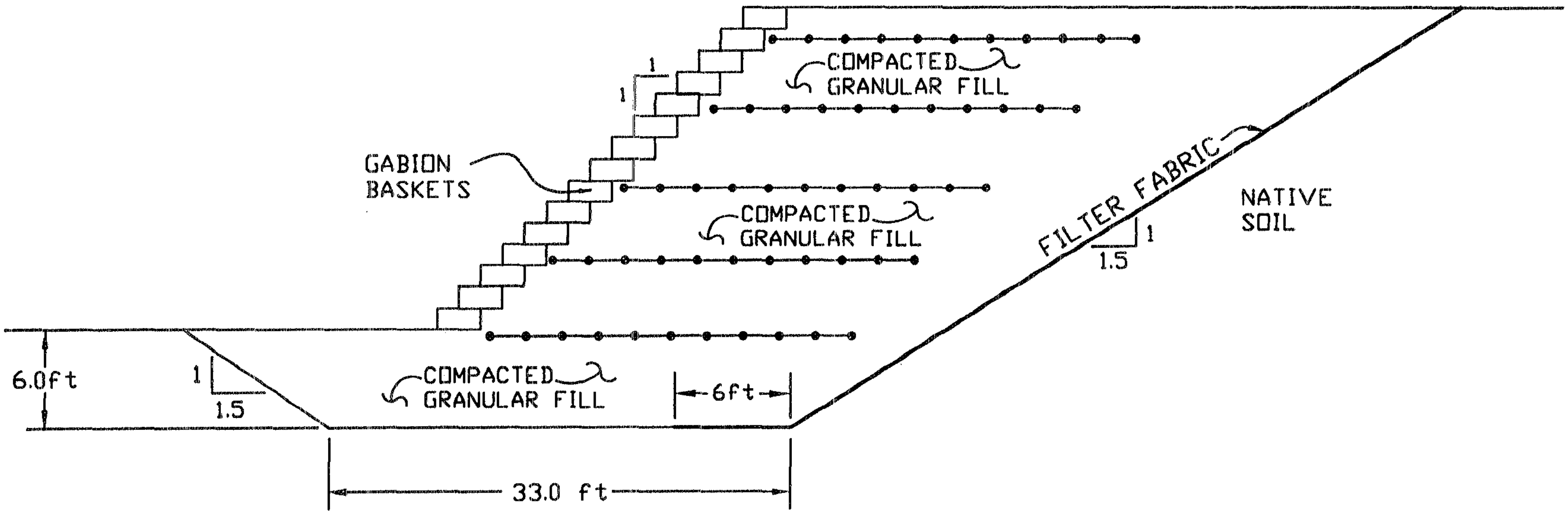
The use of crib walls will be reconsidered. Tied back sheet pile walls or the use of soil nailing should be investigated during PED. Each of those methods of ground support would permit the construction of a vertical channel wall with considerably less impact to nearby infrastructure or private property.

Site-specific investigations will be done during PED for the locations of all planned floodwalls. Then the appropriate type of floodwall will be designed and appropriate analyses performed.

A review of the final channel configurations is highly recommended to be done during PED to confirm that the slopes and elevations presented on the figures reflect the final plan cross sections for the stations analyzed.

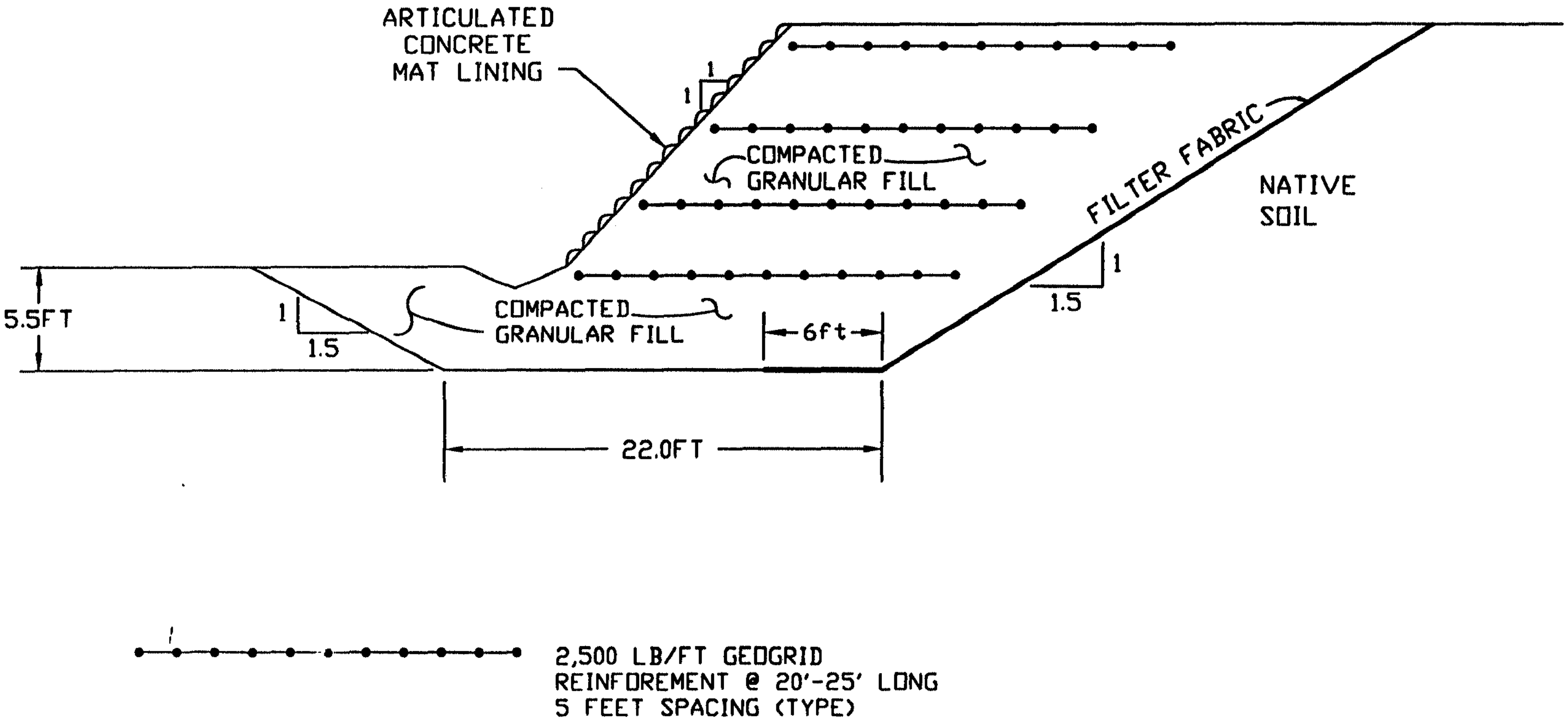


UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY LPP
TYPICAL CRIBWALL CHANNEL SECTION
REINFORCED SLOPE
FIGURE 1
NOT TO SCALE



●—●—●—●—●—●—●—●—●—●—● 2,500 LB/FT GEOGRID
REINFOREMENT @ 35' LONG
5 FEET VERTICAL SPACING (TYPE)

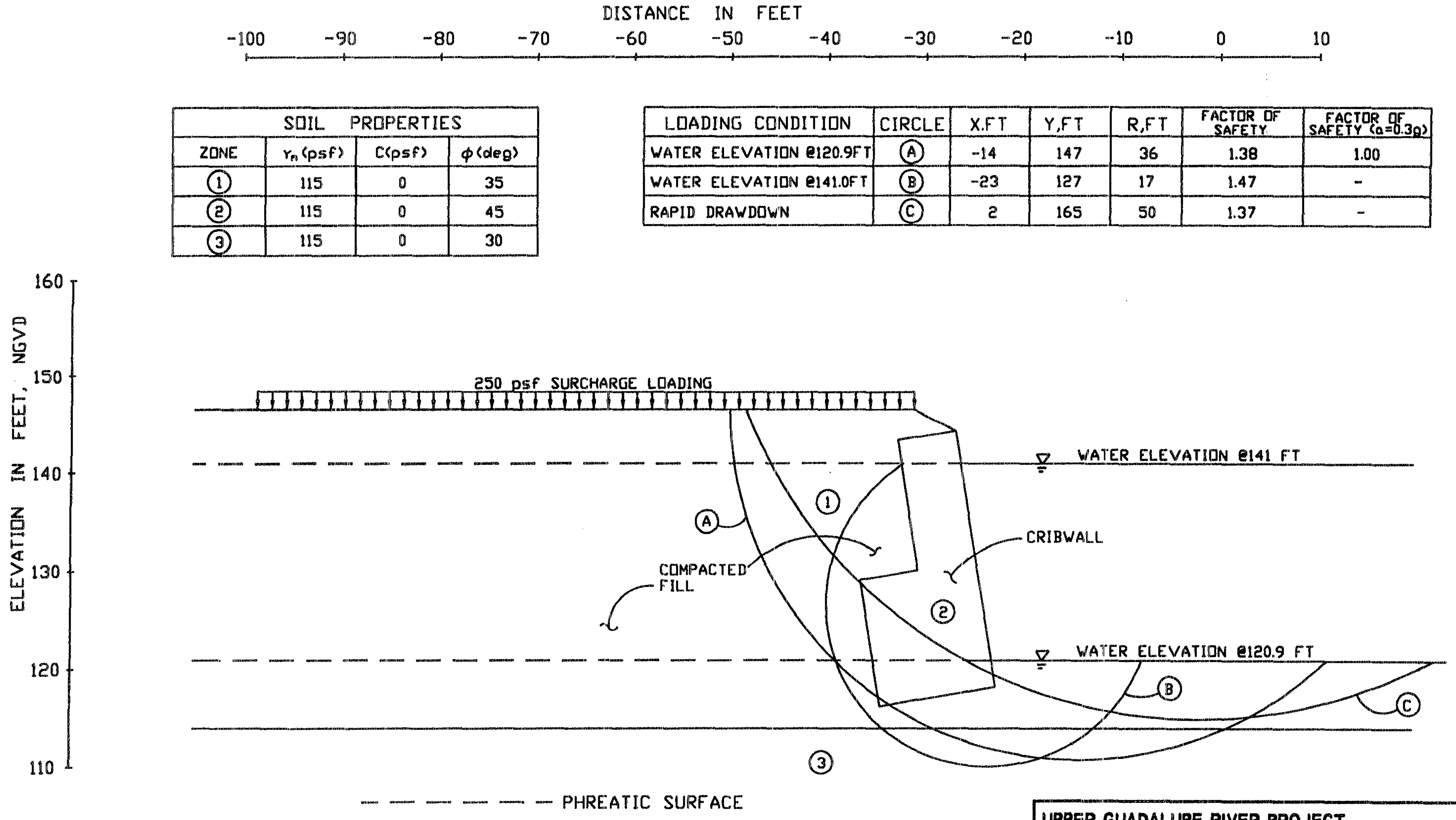
UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY LPP
TYPICAL GABION CHANNEL SECTION
REINFORCED SLOPE DETAILS
FIGURE 2
NOT TO SCALE



UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY LPP
TYPICAL ROSS CREEK CHANNEL
REINFORCED SLOPE DETAILS
FIGURE 3
NOT TO SCALE

SOIL PROPERTIES			
ZONE	γ_n (psf)	C(psf)	ϕ (deg)
①	115	0	35
②	115	0	45
③	115	0	30

LOADING CONDITION	CIRCLE	X,FT	Y,FT	R,FT	FACTOR OF SAFETY	FACTOR OF SAFETY (a=0.3g)
WATER ELEVATION @120.9FT	Ⓐ	-14	147	36	1.38	1.00
WATER ELEVATION @141.0FT	Ⓑ	-23	127	17	1.47	-
RAPID DRAWDOWN	Ⓒ	2	165	50	1.37	-

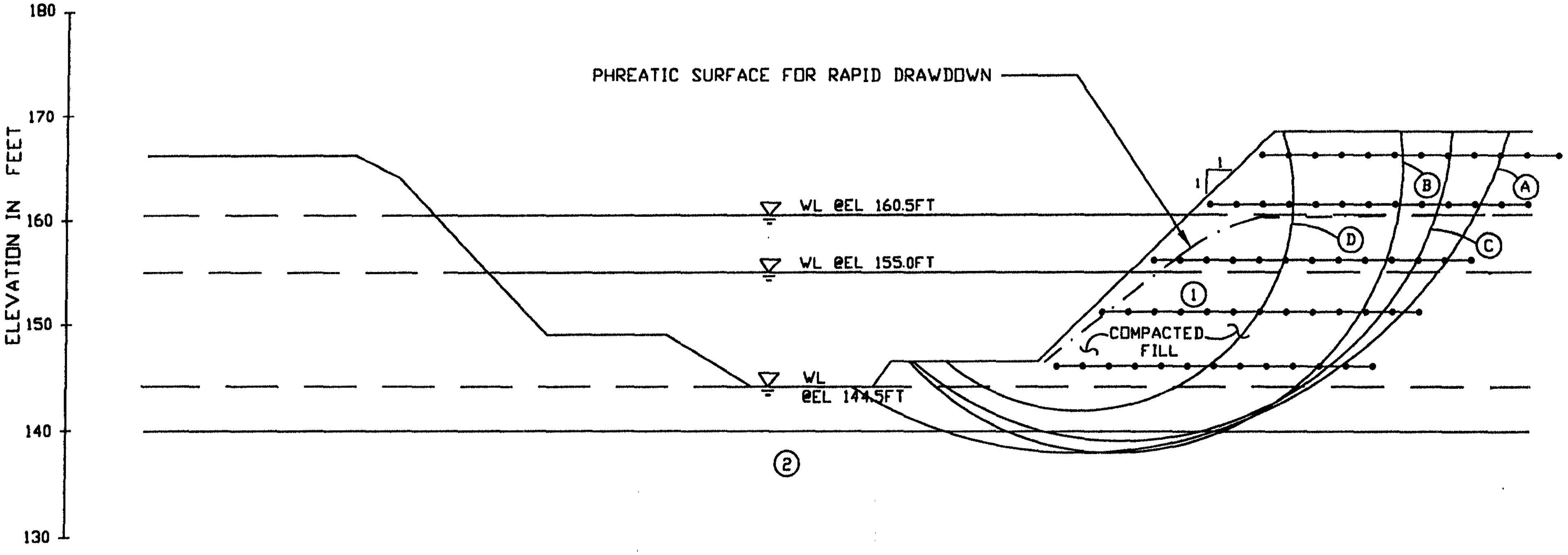


- - - - - - PHREATIC SURFACE

UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY
SLOPE STABILITY ANALYSIS WITH COMPACTED FILL
STATION 854+00
LOOKING UPSTREAM
BYPASS CHANNEL PLAN
FIGURE 4

SOIL PROPERTIES			
ZONE	γ_n (psf)	C(psf)	ϕ (deg)
①	123	0	35
②	123	0	30

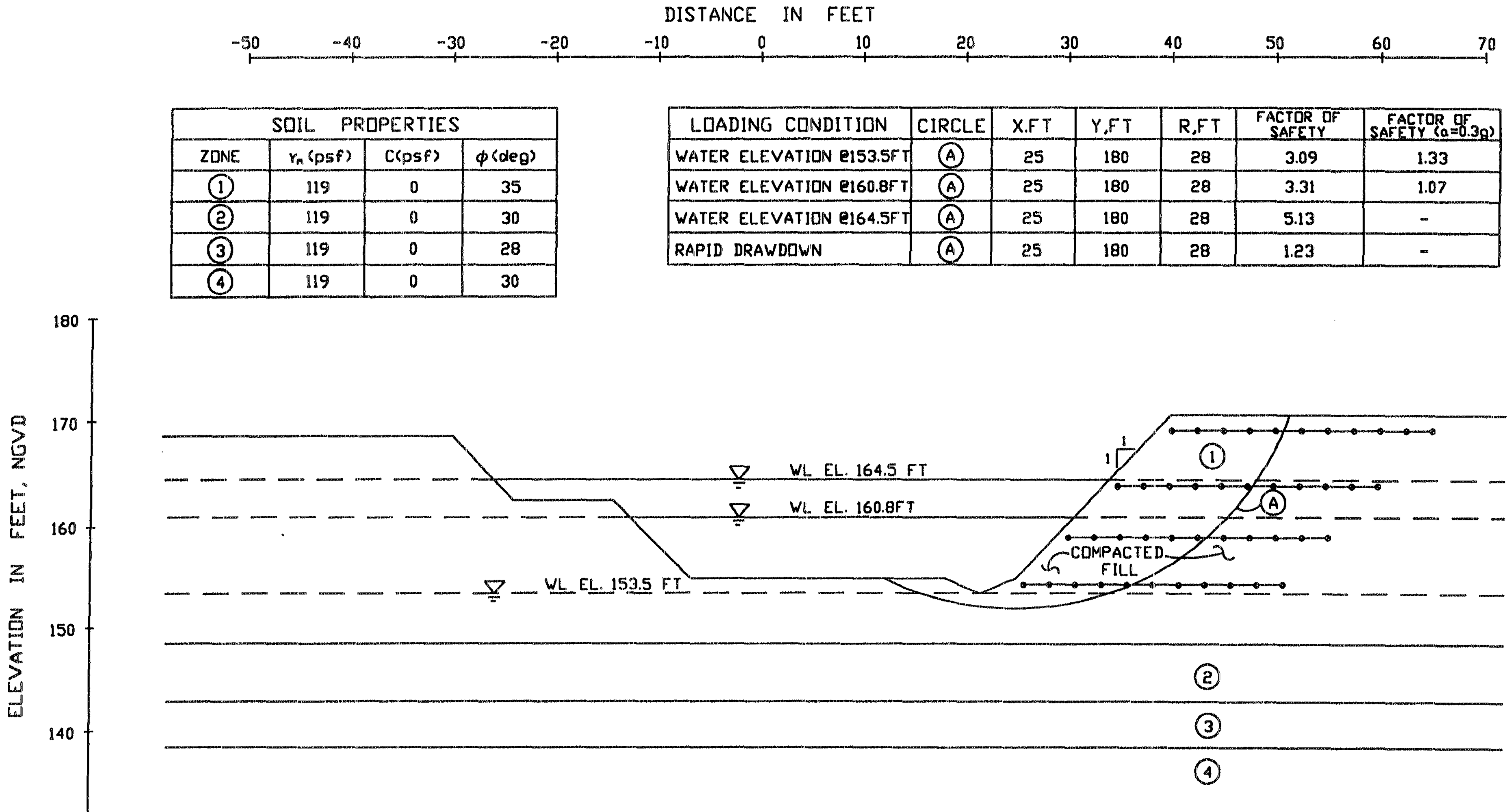
LOADING CONDITION	CIRCLE	X,FT	Y,FT	R,FT	FACTOR OF SAFETY	FACTOR OF SAFETY (a=0.3g)
WATER ELEVATION @144.5FT	Ⓐ	40	180	42	2.00	1.02
WATER ELEVATION @155.0FT	Ⓑ	43	165	27	1.80	1.08
WATER ELEVATION @160.5FT	Ⓒ	44	170	31	1.90	-
RAPID DRAWDOWN	Ⓓ	40	162	20	1.30	-



—●—●—●— 2,500 LB/FT GEOGRID REINFOREMENT @ 30'-35' LONG 5 FOOT VERTICAL SPACING (TYPE)

— — — PHREATIC SURFACE

UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY
SLOPE STABILITY ANALYSIS WITH REINFORCEMENT
STATION 936+00
LOOKING UPSTREAM
BYPASS CHANNEL PLAN
FIGURE 5

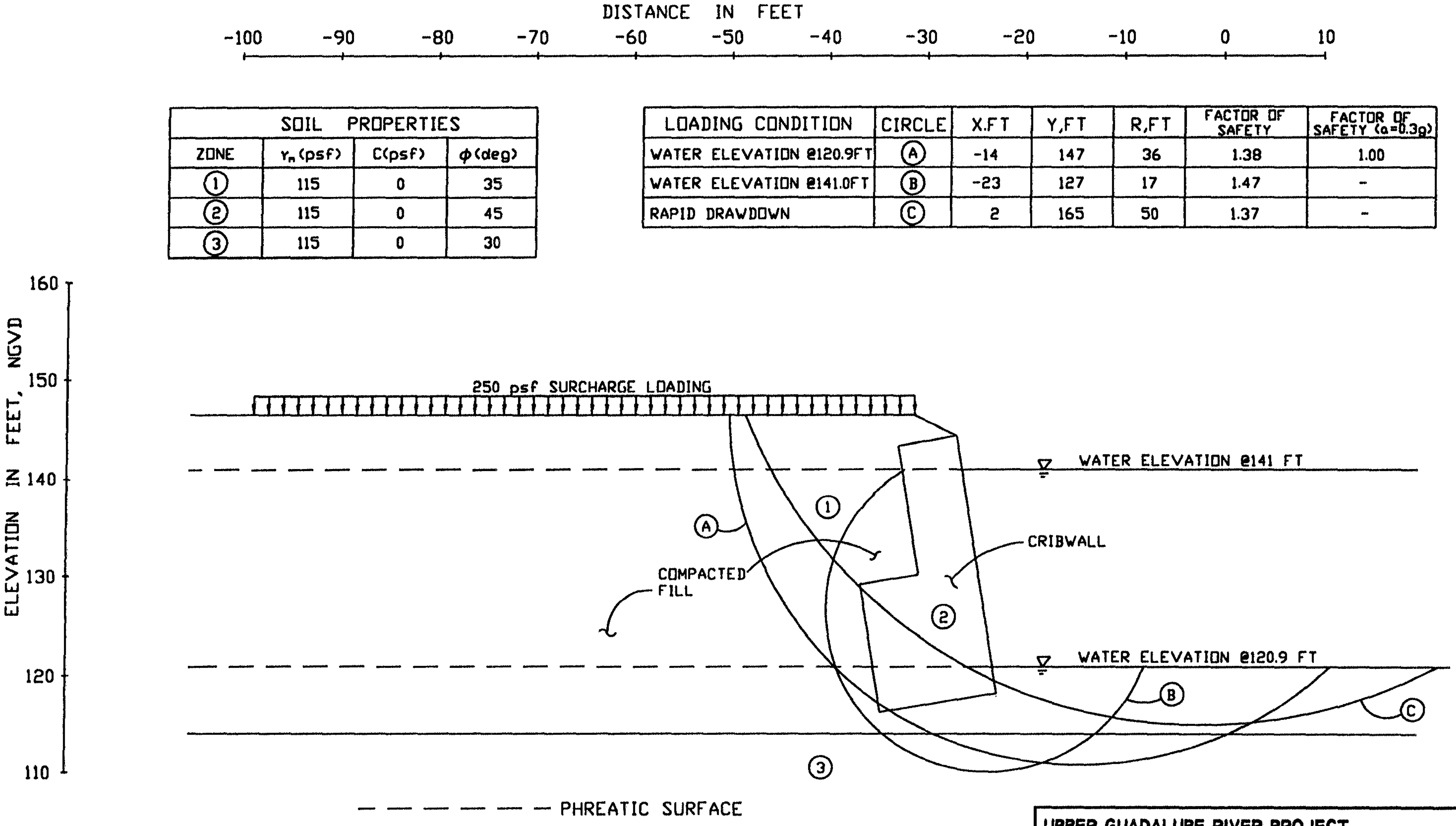


SOIL PROPERTIES			
ZONE	γ_m (psf)	C(psf)	ϕ (deg)
①	119	0	35
②	119	0	30
③	119	0	28
④	119	0	30

LOADING CONDITION	CIRCLE	X,FT	Y,FT	R,FT	FACTOR OF SAFETY	FACTOR OF SAFETY (a=0.3g)
WATER ELEVATION @153.5FT	Ⓐ	25	180	28	3.09	1.33
WATER ELEVATION @160.8FT	Ⓐ	25	180	28	3.31	1.07
WATER ELEVATION @164.5FT	Ⓐ	25	180	28	5.13	-
RAPID DRAWDOWN	Ⓐ	25	180	28	1.23	-

●—●—●—●—●—●—●—●—●—●—●	2,500 LB/FT GEOGRID REINFOREMENT @ 20'-25' LONG 5 FOOT VERTICAL SPACING (TYPE)
— — — — — — —	PHREATIC SURFACE

UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY
SLOPE STABILITY ANALYSIS WITH REINFORCEMENT
STATION 9+00 ROSS CREEK
LOOKING UPSTREAM
BYPASS CHANNEL PLAN
FIGURE 6



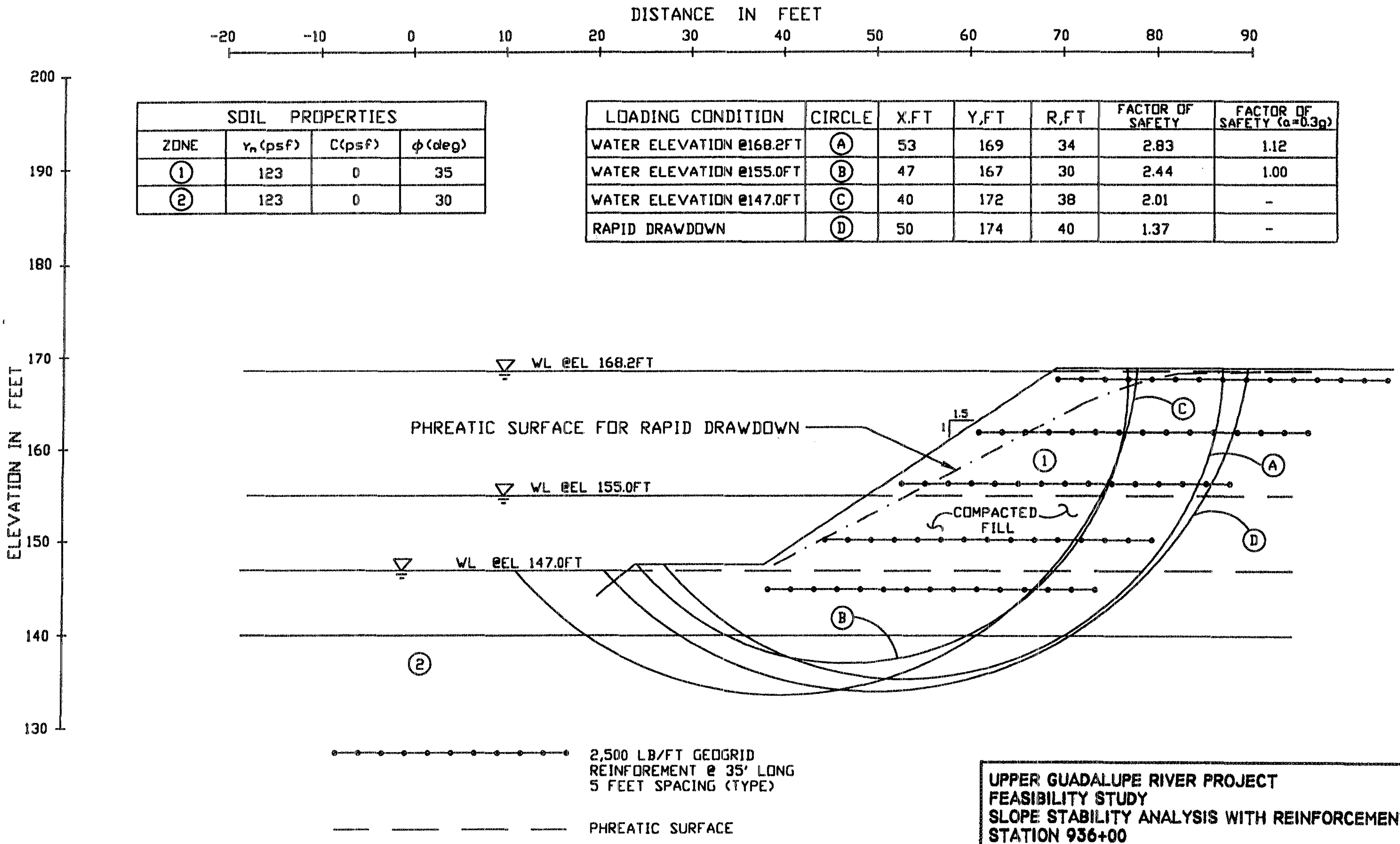
SOIL PROPERTIES			
ZONE	γ_m (psf)	C(psf)	ϕ(deg)
①	115	0	35
②	115	0	45
③	115	0	30

LOADING CONDITION	CIRCLE	X.FT	Y,FT	R,FT	FACTOR OF SAFETY	FACTOR OF SAFETY (a=0.3g)
WATER ELEVATION @120.9FT	Ⓐ	-14	147	36	1.38	1.00
WATER ELEVATION @141.0FT	Ⓑ	-23	127	17	1.47	-
RAPID DRAWDOWN	Ⓒ	2	165	50	1.37	-

UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY
SLOPE STABILITY ANALYSIS WITH COMPACTED FILL
STATION 854+00
LOOKING UPSTREAM
VALLEY VIEW PLAN
FIGURE 7

SOIL PROPERTIES			
ZONE	γ_n (psf)	C(psf)	ϕ (deg)
①	123	0	35
②	123	0	30

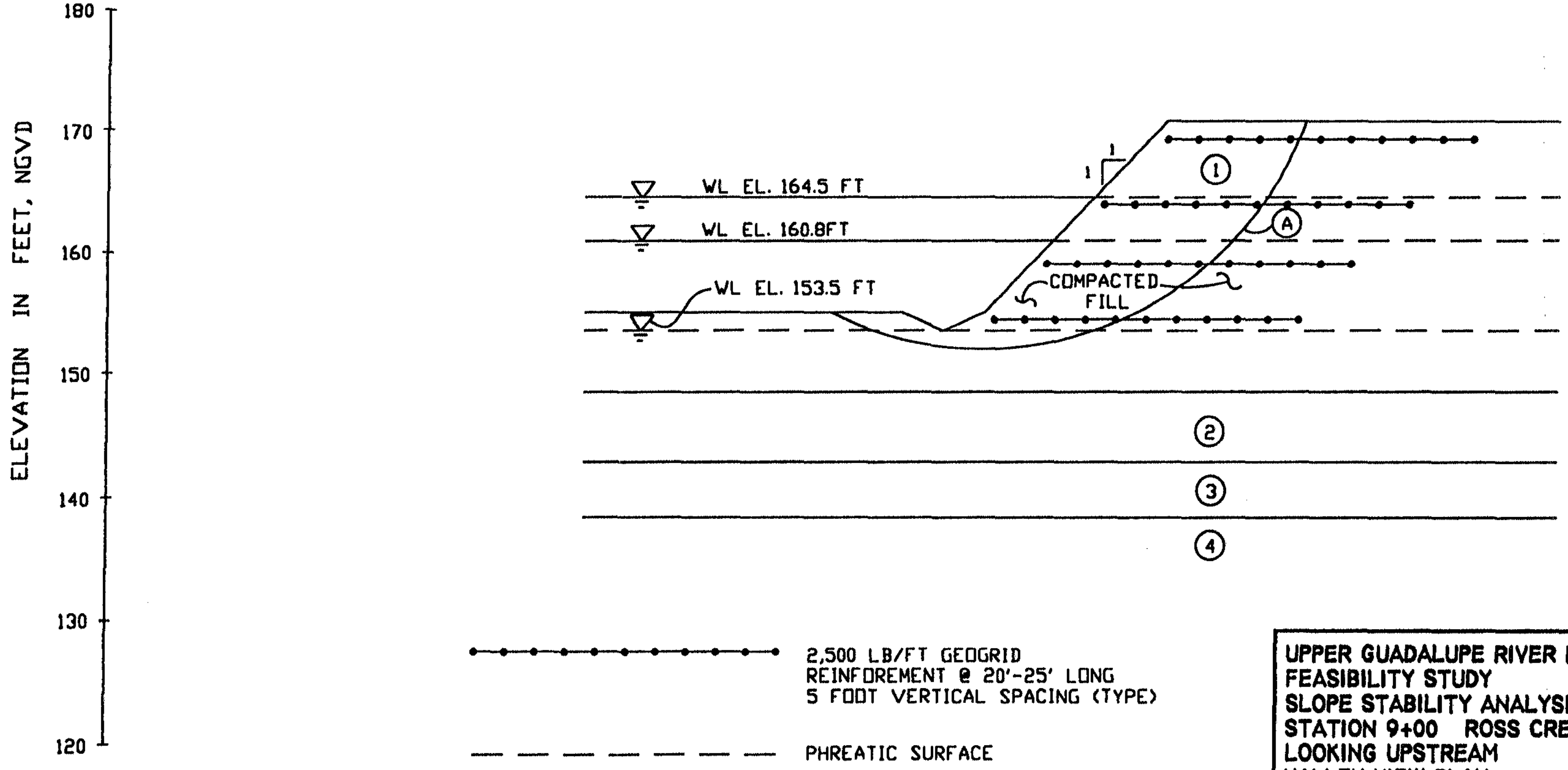
LOADING CONDITION	CIRCLE	X,FT	Y,FT	R,FT	FACTOR OF SAFETY	FACTOR OF SAFETY (a=0.3g)
WATER ELEVATION @168.2FT	Ⓐ	53	169	34	2.83	1.12
WATER ELEVATION @155.0FT	Ⓑ	47	167	30	2.44	1.00
WATER ELEVATION @147.0FT	Ⓒ	40	172	38	2.01	-
RAPID DRAWDOWN	Ⓓ	50	174	40	1.37	-



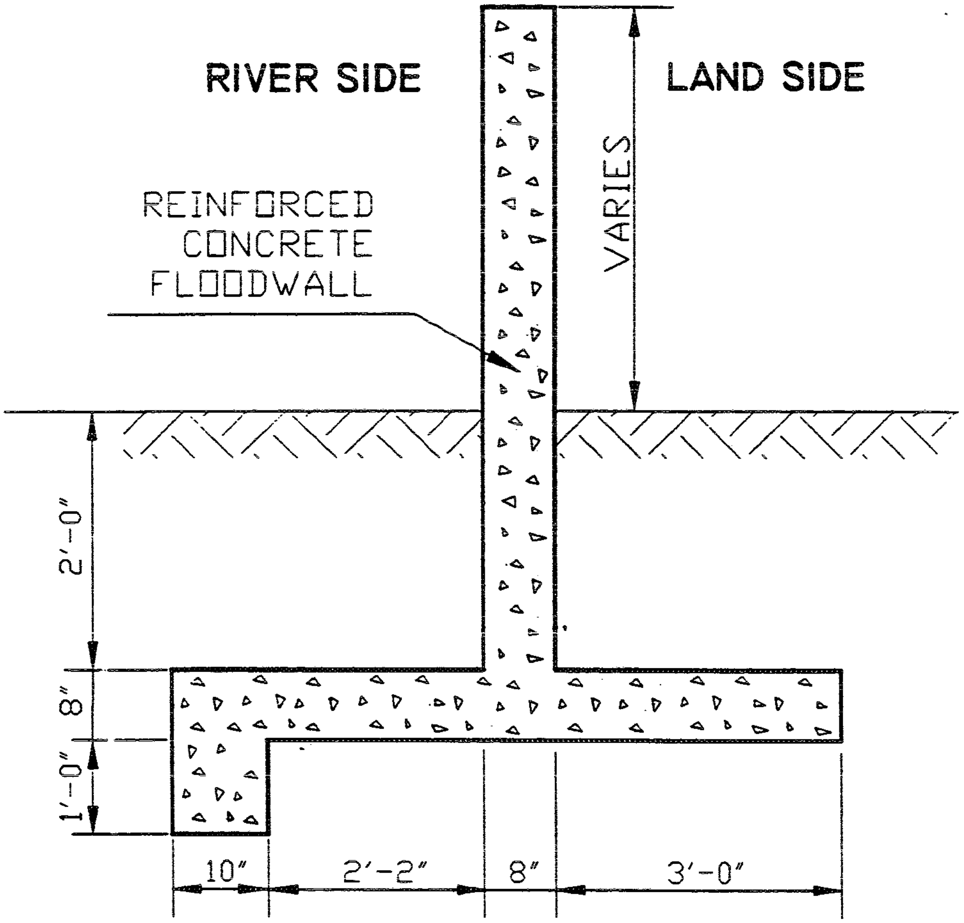
UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY
SLOPE STABILITY ANALYSIS WITH REINFORCEMENT
STATION 936+00
LOOKING UPSTREAM
VALLEY VIEW PLAN
FIGURE 8

SOIL PROPERTIES			
ZONE	γ_m (psf)	C(psf)	ϕ (deg)
①	119	0	35
②	119	0	30
③	119	0	28
④	119	0	30

LOADING CONDITION	CIRCLE	X,FT	Y,FT	R,FT	FACTOR OF SAFETY	FACTOR OF SAFETY (a=0.3g)
WATER ELEVATION @153.5FT	Ⓐ	25	180	28	3.09	1.33
WATER ELEVATION @160.8FT	Ⓐ	25	180	28	3.31	1.07
WATER ELEVATION @164.5FT	Ⓐ	25	180	28	5.13	-
RAPID DRAWDOWN	Ⓐ	25	180	28	1.23	-



UPPER GUADALUPE RIVER PROJECT
FEASIBILITY STUDY
SLOPE STABILITY ANALYSIS WITH REINFORCEMENT
STATION 9+00 ROSS CREEK
LOOKING UPSTREAM
VALLEY VIEW PLAN
FIGURE 9



TYPICAL FLOODWALL

US ARMY ENGINEER DISTRICT SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA
DRAWN BY: SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
PROPOSED FLOODWALL

CHECKED BY:
APPROVED: DATE:
SCALE: NTS JOB NO:
DRAWING NO:
SHEET: FIGURE: 10

ATTACHMENT A

PROFILE OF TIP RESISTANCE

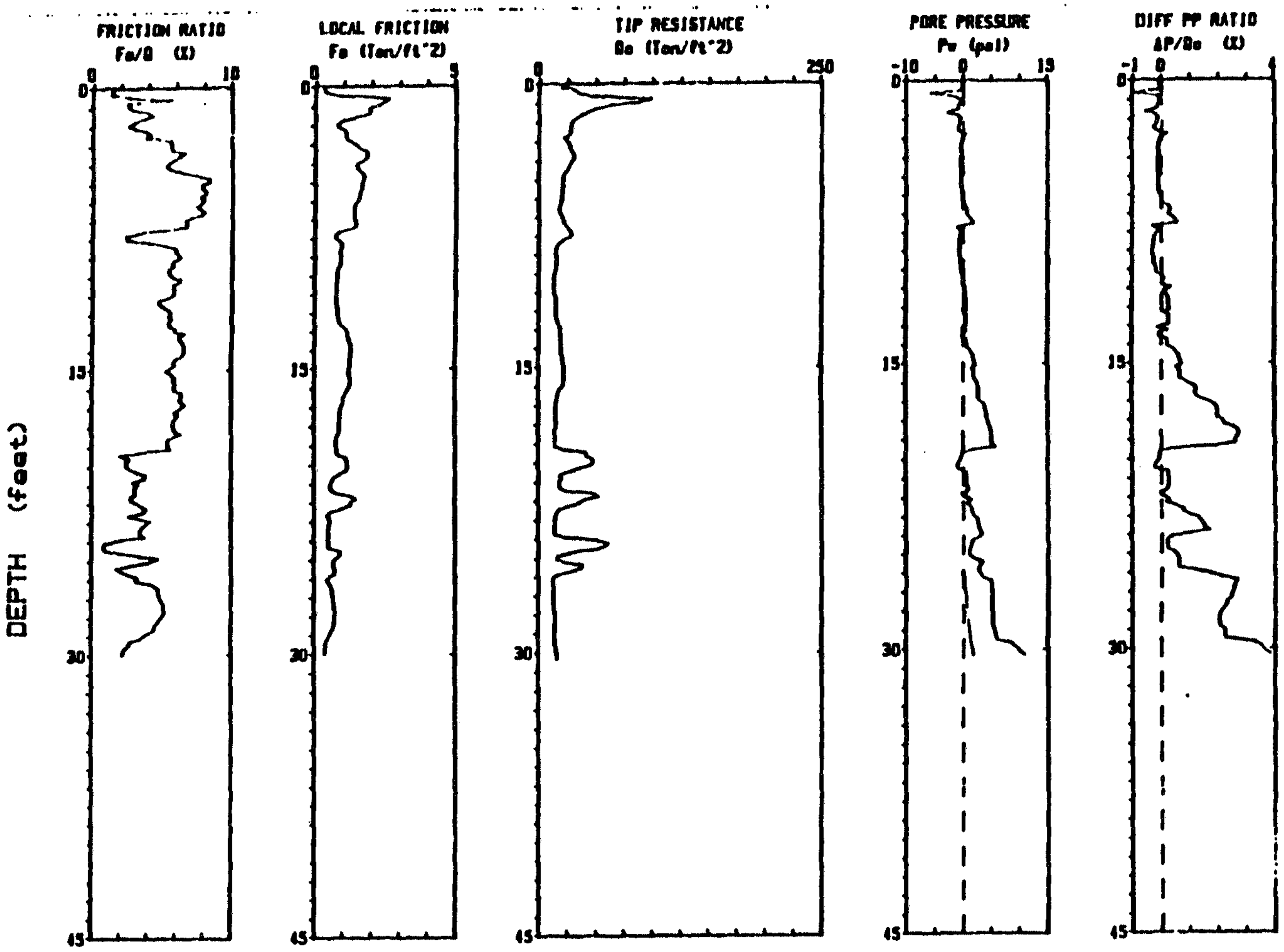
CPT TEST DATA

UPPER GUADALUPE RIVER PROJECT

Operator : CARL G. MORGAN
Location : GCPT-1

CPT Date : 02-27-92 08:20
Cone Used : 347TC

Sounding : 920070 Pg 1 / 1
Job No. : 0086-045 -003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

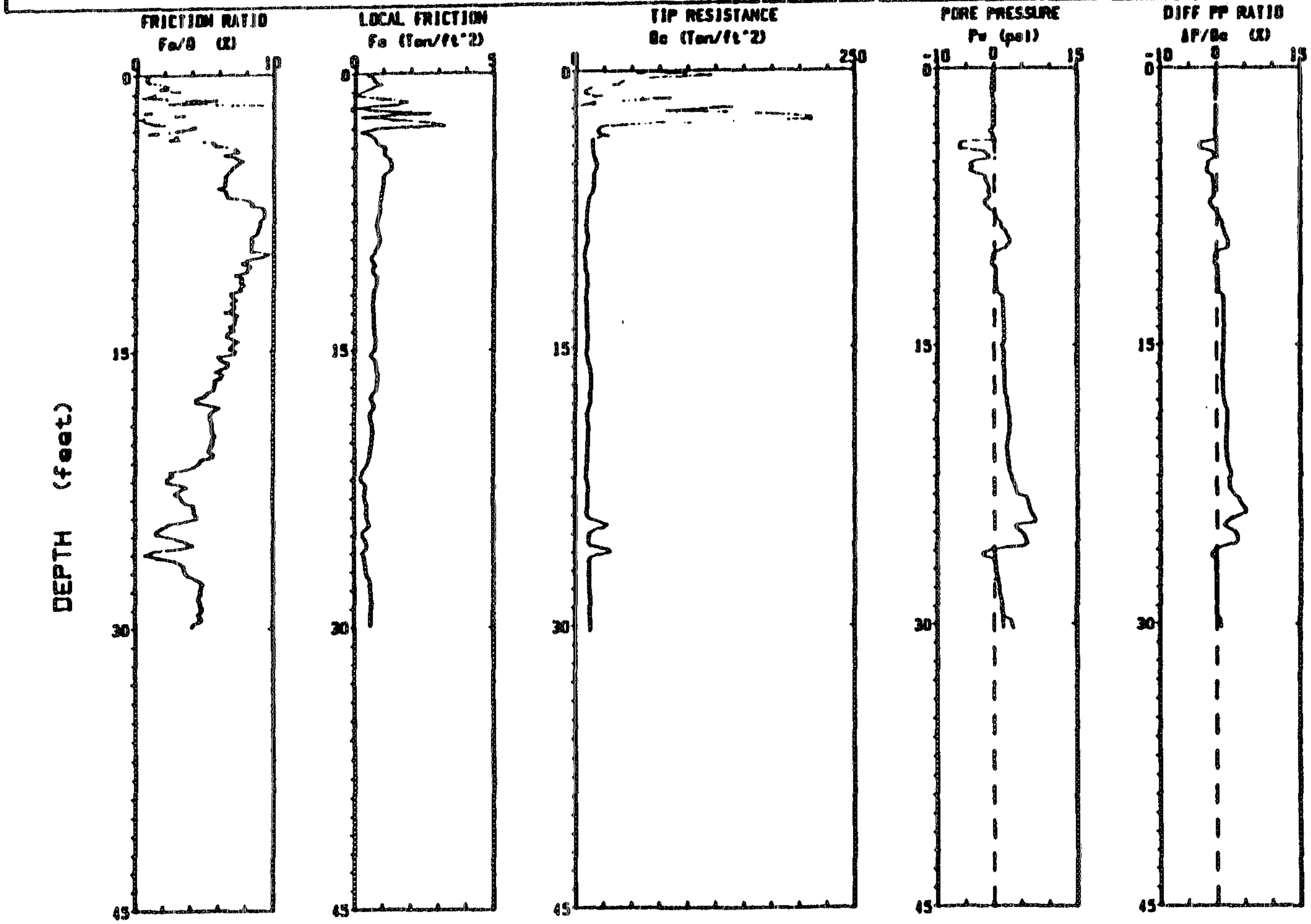
CHECKED BY ____________ FILE 00086-045-003 ____________ BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-2

CPT Date : 02-27-82 09:44
Cone Used : 347TC

Sounding : 820071 Pg 1 / 1
Job No. : 00086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

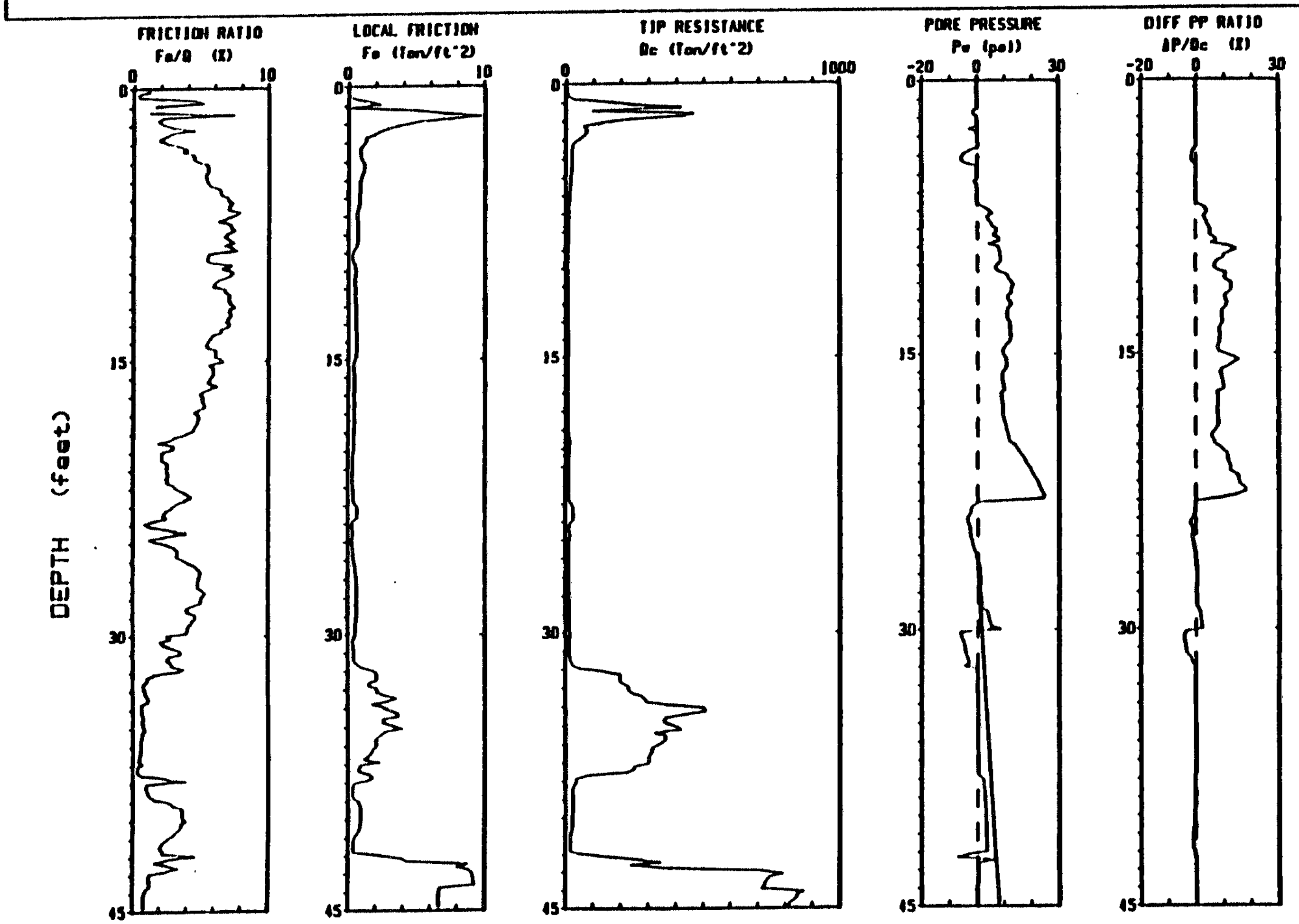
A-2

V B I

Operator : CARL G. MORGAN
Location : GCPT-3

CPT Date : 03-12-92 11:37
Cone Used : 347TC

Sounding : 920114 Pg 1
Job No. : 0088-045-003



Depth Increment : .05 m

Max Depth : 45.11 ft

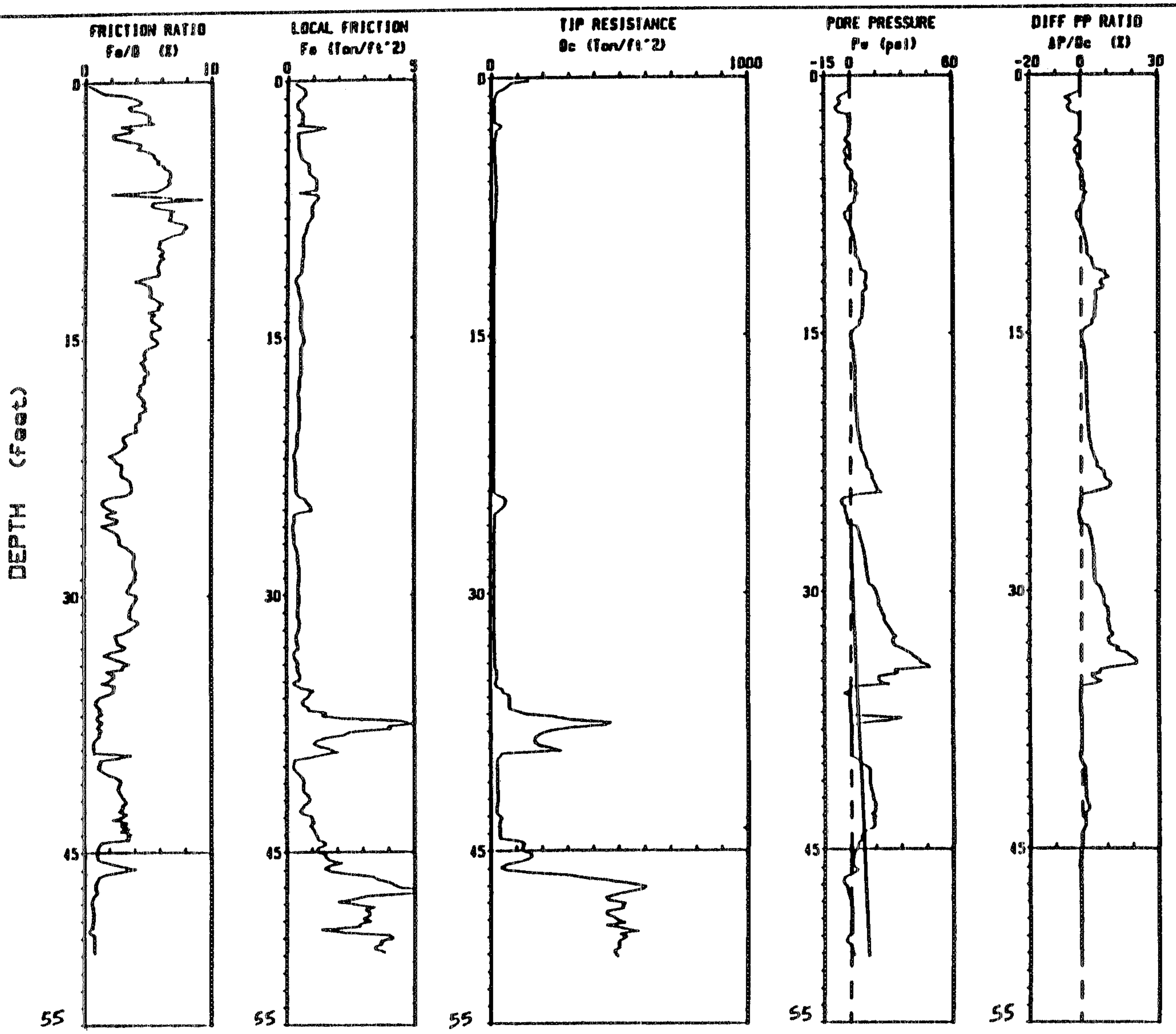
CHECKED BY ______________ FILE 00086-045-003 ______________ BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-4

CPT Date : 03-12-92 09:17
Cone Used : 347TC

Sounding : 920113 Pg 1
Job No. : 0086-045-003



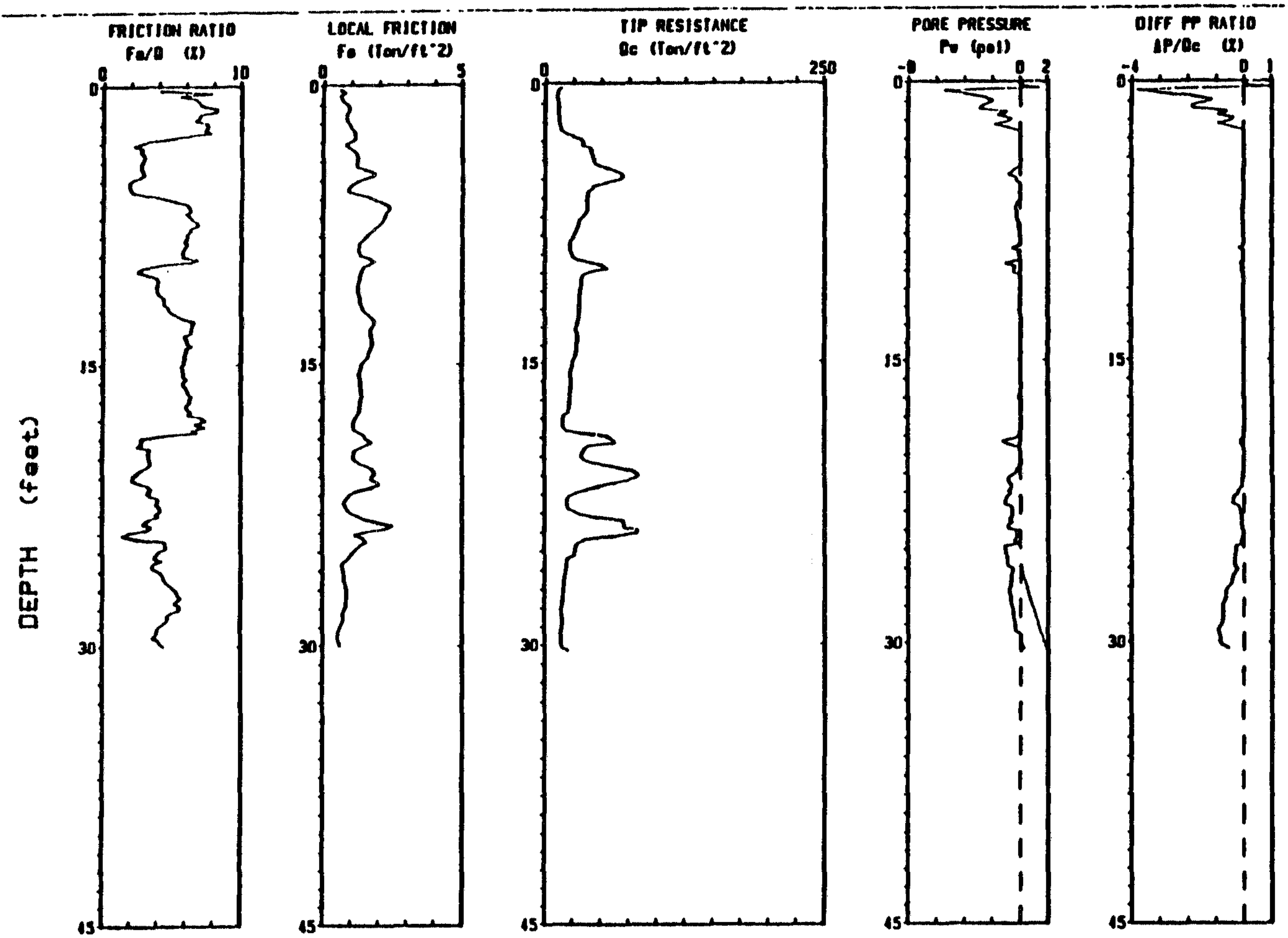
Max. Depth = 51.18 ft

V B I

Operator : CARL G. MORGAN
Location : GCPT-5

CPT Date : 02-25-92 08:57
Cone Used : 347TC

Sounding : 920061 Pg 1 / 1
Job No. : 00086-045-003



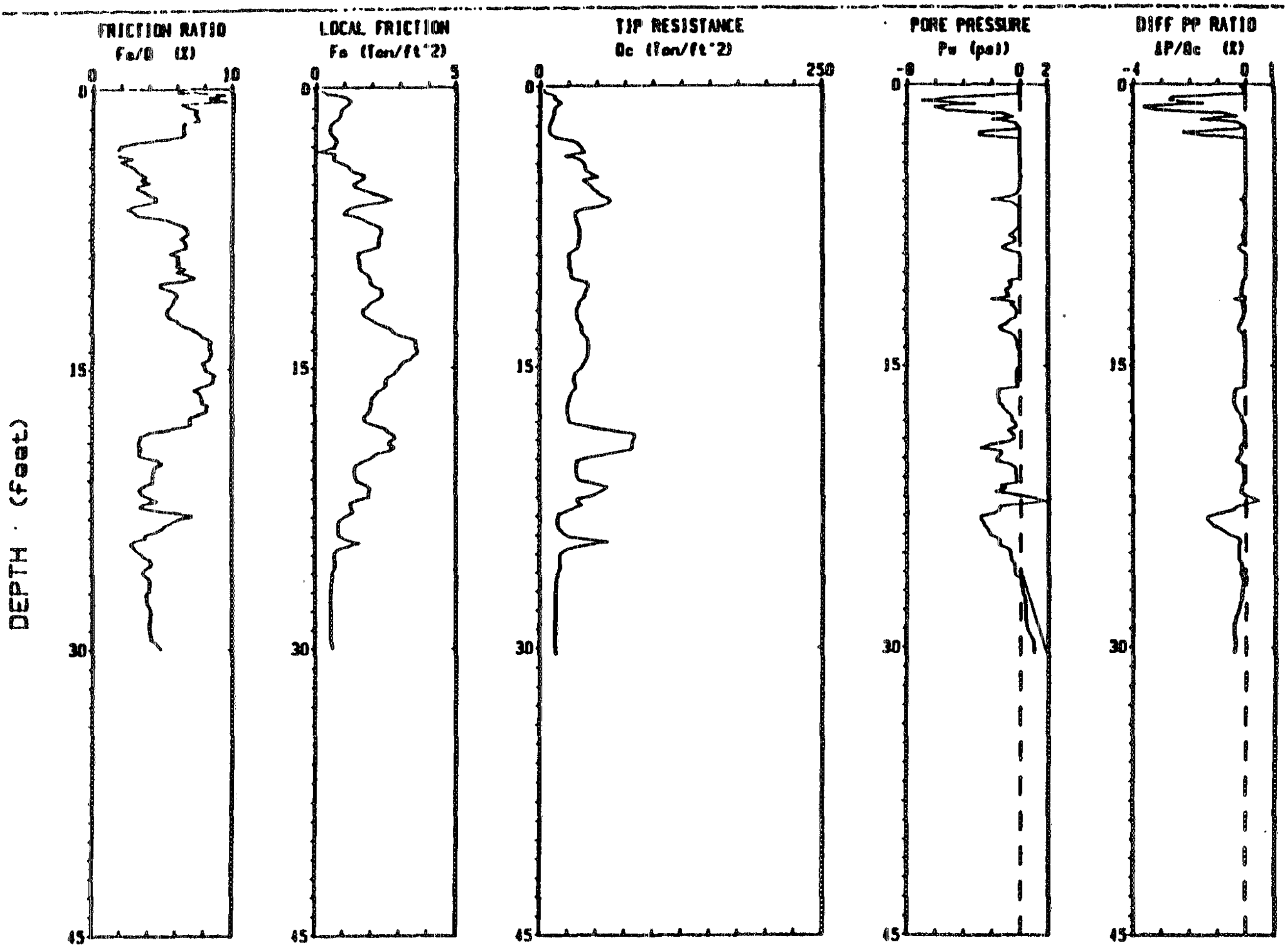
Depth Increment : .05 m

Max Depth : 30.35 ft

CHECKED BY ____________ FILE 00086- 045-003 ____________ REVISIONS BY ________ DATE ______

V B I

Operator : CARL G. MORGAN CPT Date : 02-25-92 10:30 Sounding : 920062 Pg 1 / 1
Location : GCPT-6 Cone Used : 347TC Job No. : 00086-045-003



Depth Increment : .05 m Max Depth : 30.35 ft

Dames & Moore

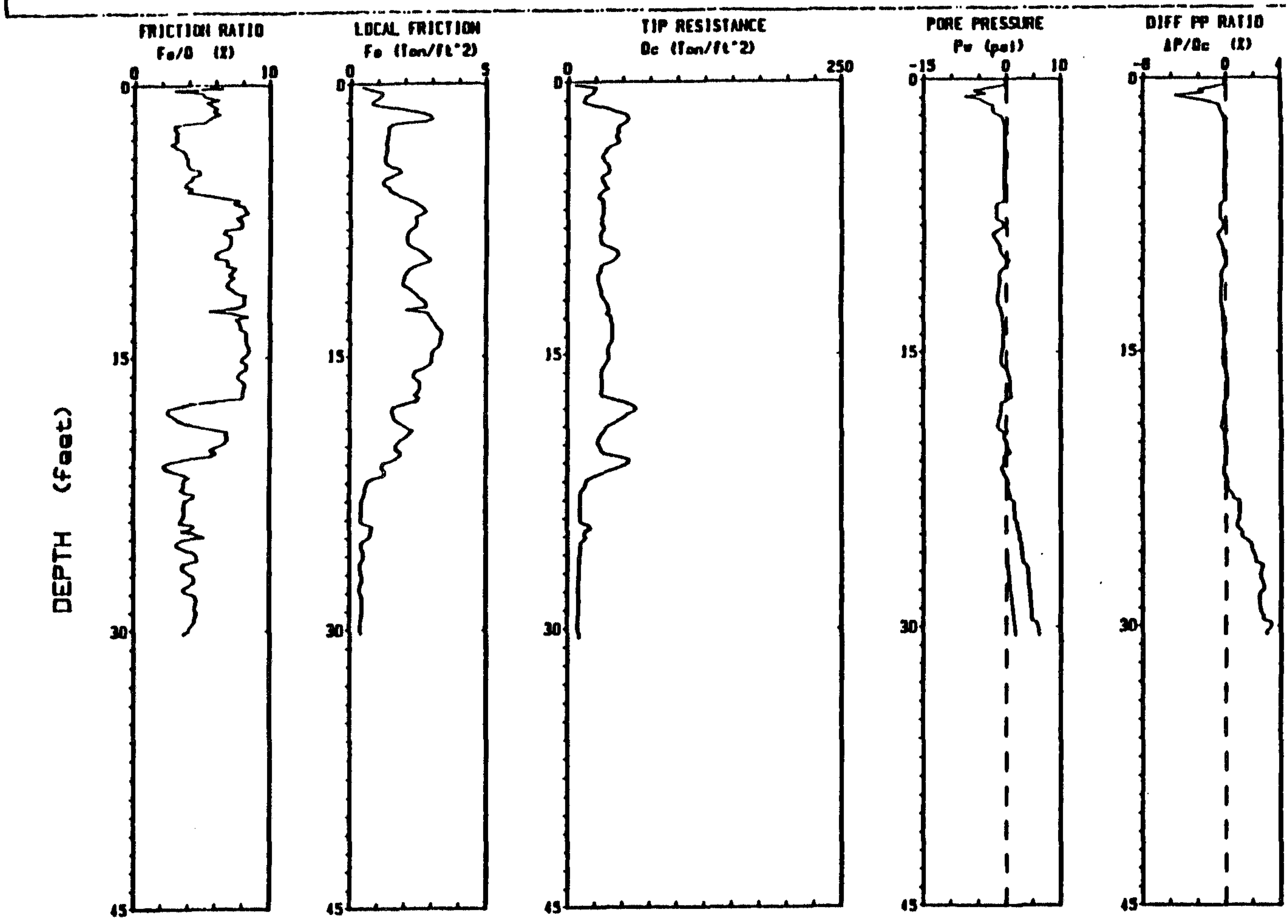
#-6

V B I

Operator : CARL G. MORGAN
Location : DP-1

CPT Date : 02-25-92 11:41
Cone Used : 347TC

Sounding : 920083 Pg 1 / 1
Job No. : 00086-045-003



Depth Increment : .05 m

Max Depth : 30.51 ft

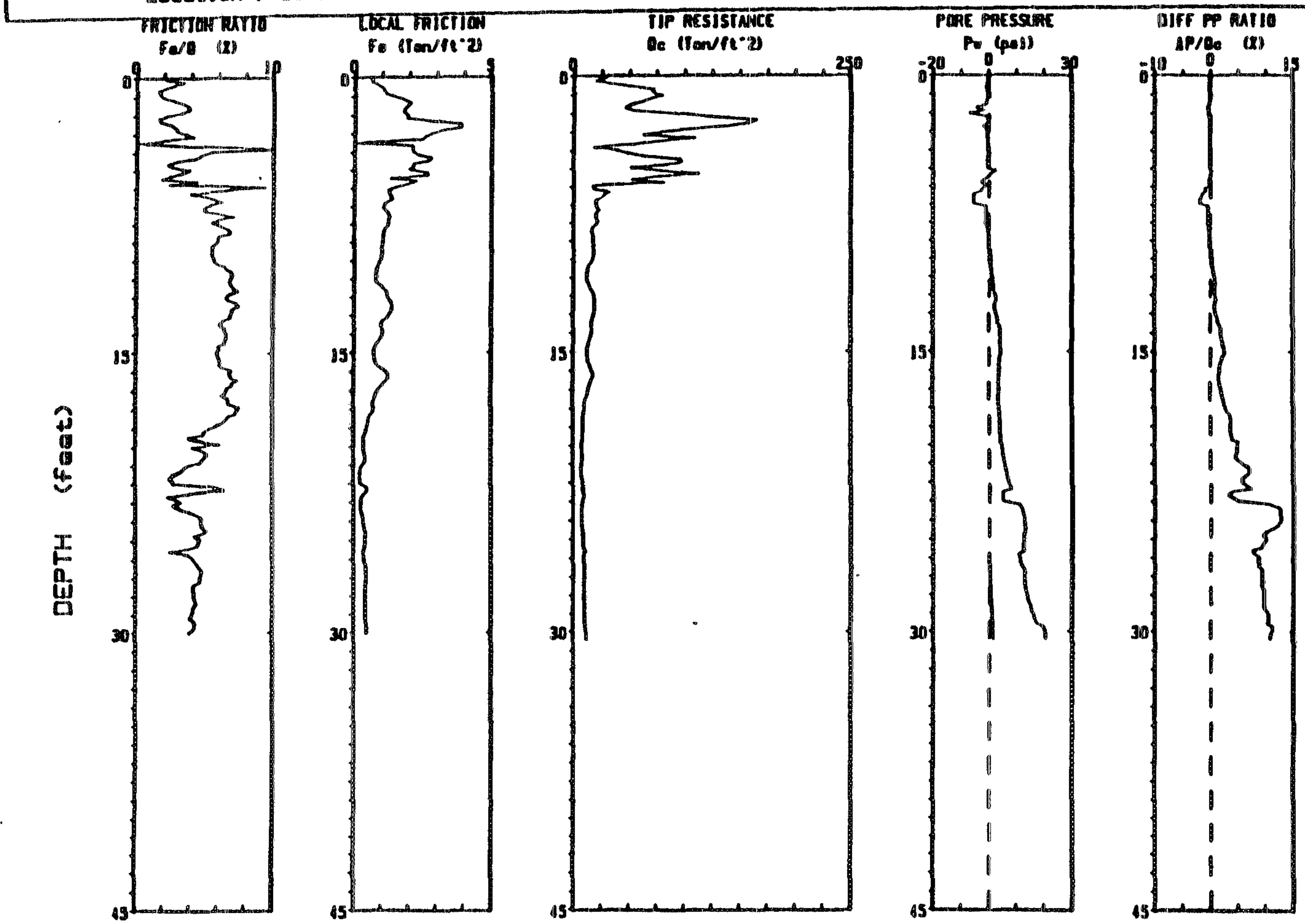
CHECKED BY ____________ FILE 00086-045-003 BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-7

CPT Date : 02-25-92 13:00
Cone Used : 347TC

Sounding : 920064 Pg 1 / 1
Job No. : 00086-045-003



Depth Increment : .05 m

Max Depth : 30.91 ft

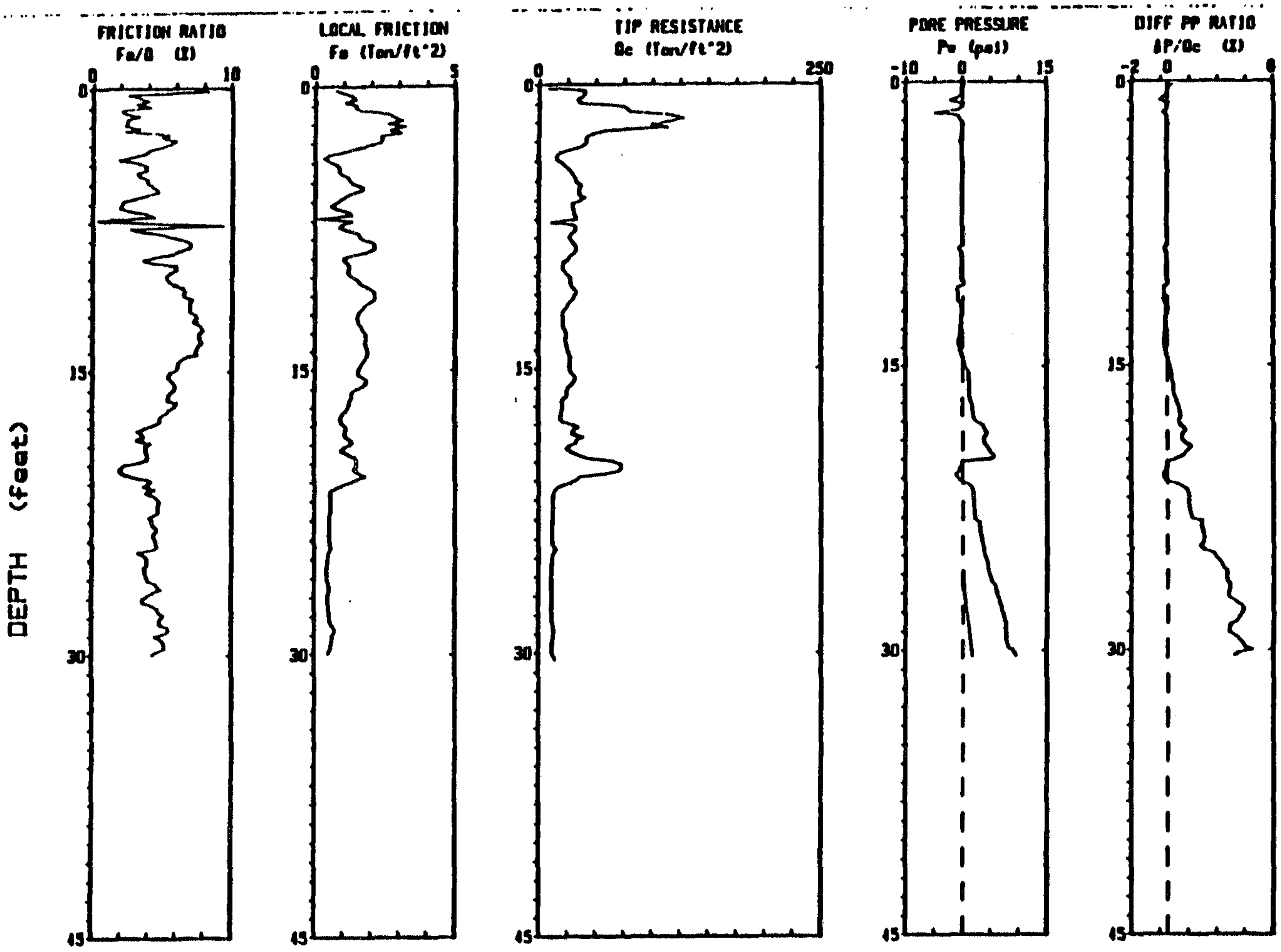
Dames & Moore

V B I

Operator : CARL G. MORGAN
Location : DP-2

CPT Date : 02-26-82 08:14
Cone Used : 347TC

Sounding : 820065 Pg 1 / 1
Job No. : 00086-045 -003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

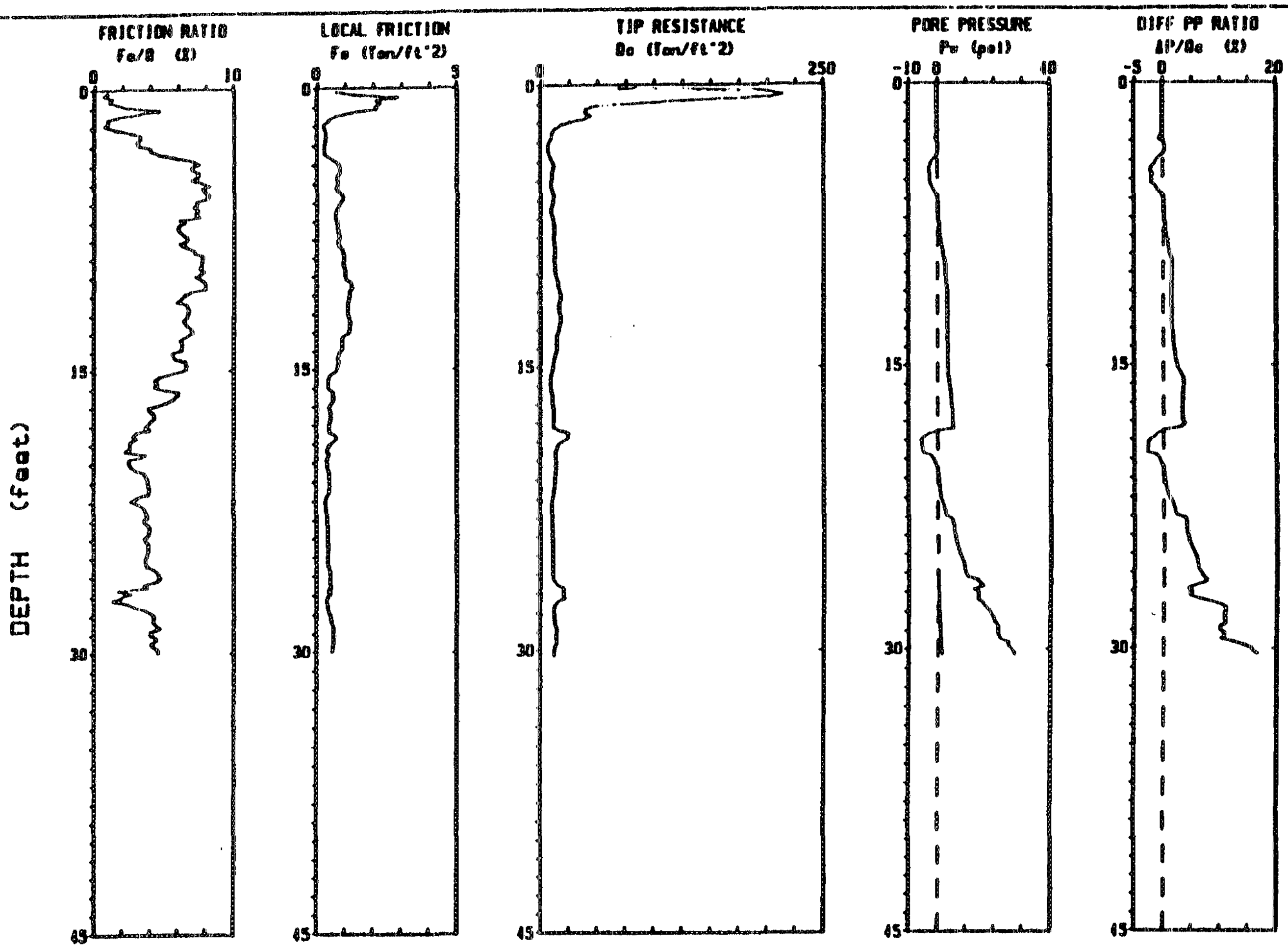
CHECKED BY ______ FILE 00086-045-003 BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-8

CPT Date : 02-26-92 08:38
Cone Used : 347TC

Sounding : 920066 Pg 1 / 1
Job No. : 00086-045-003



Depth Increment : .05 m

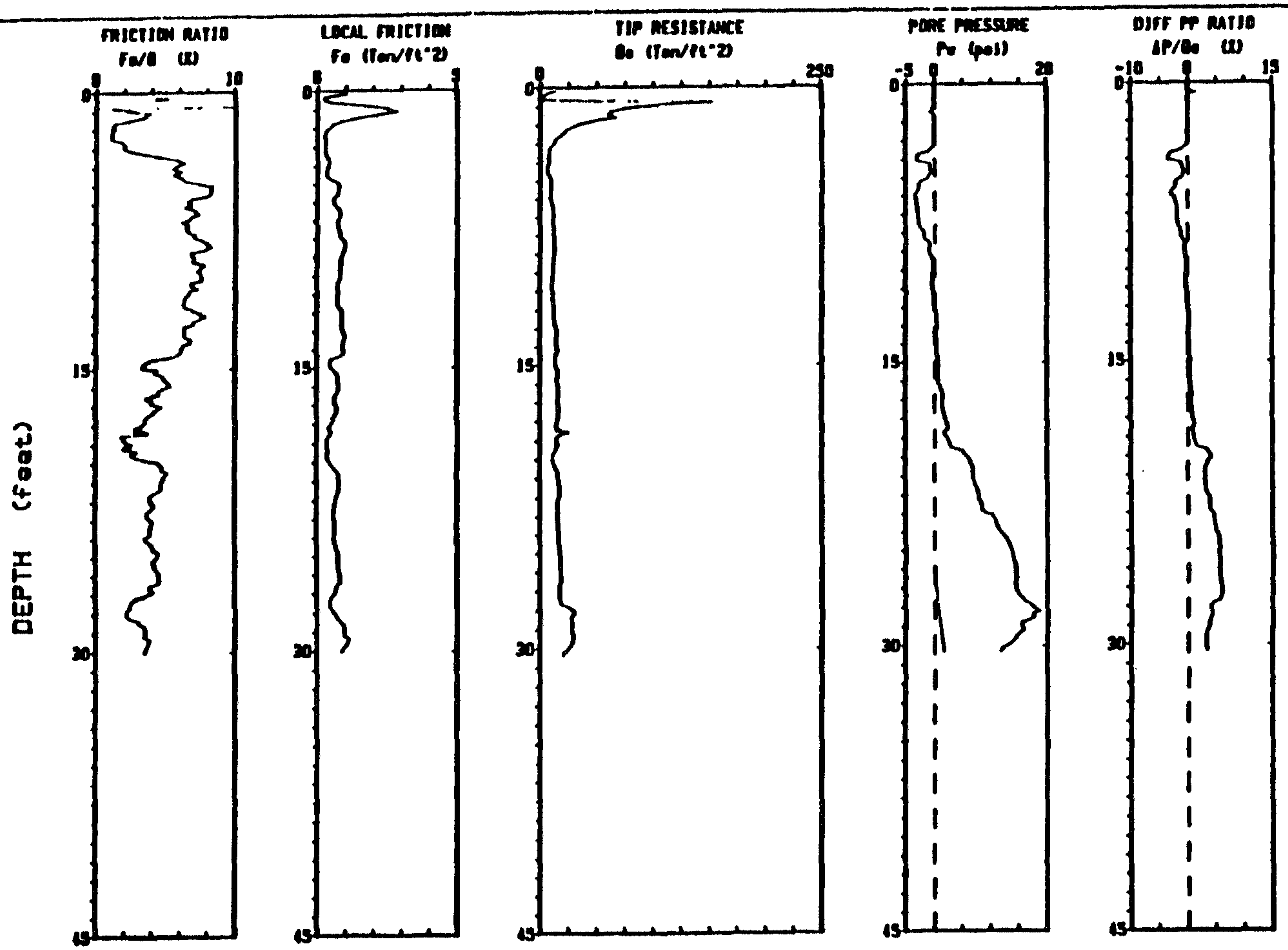
Max Depth : 30.35 ft

V B I

Operator : CARL G. MORGAN
Location : DP-3

CPT Date : 02-27-92 11:10
Cone Used : 347TC

Sounding : 920072 Pg 1 / 1
Job No. : 00086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

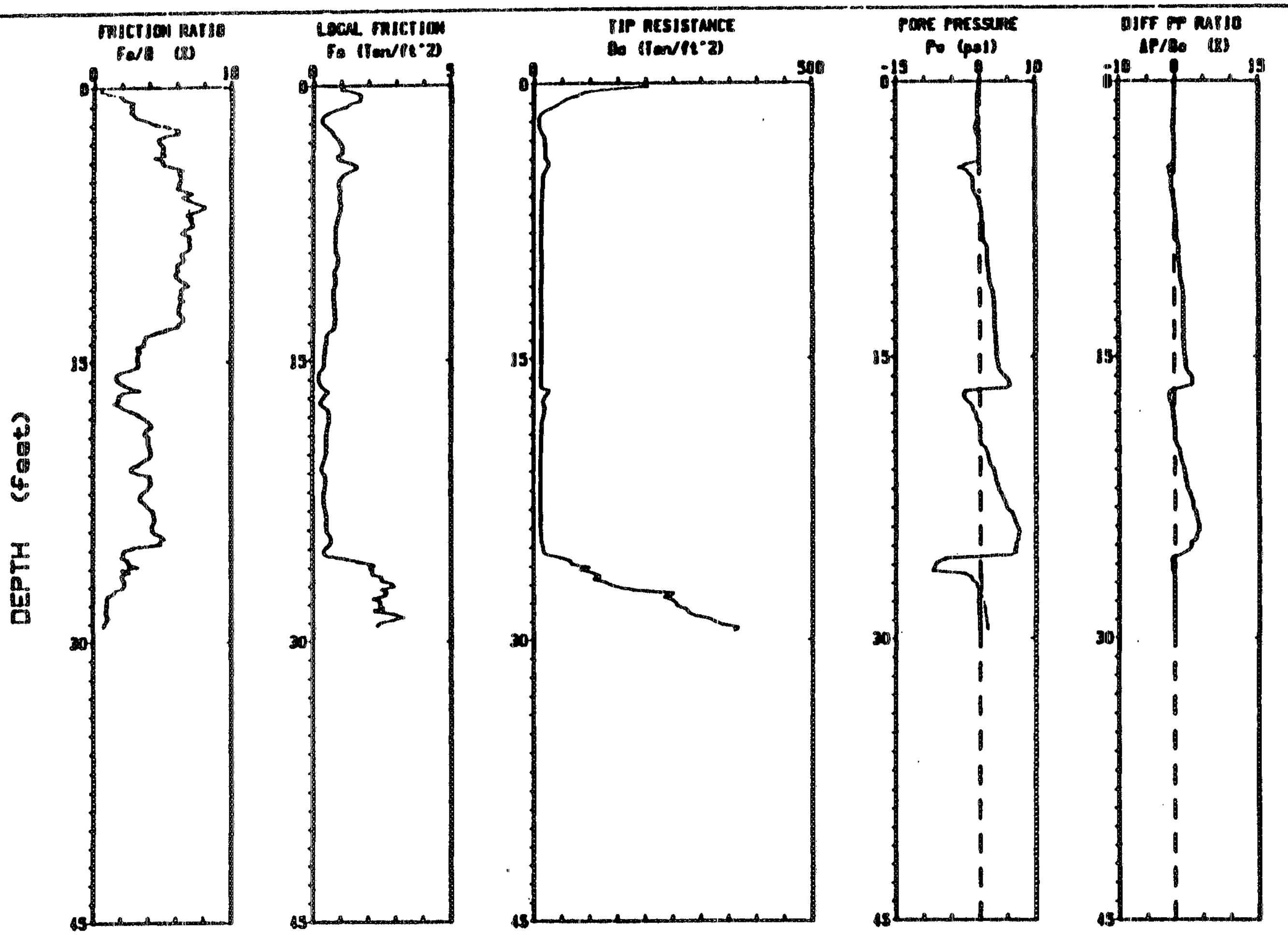
Dames & Moore

CHECKED BY ____________ FILE 00086-045-003 BY ________ DATE ______

V B I

Operator : CARL G. MORGAN　CPT Date : 02-27-82 12:15　Sounding : 820073 Pg 1 / 1

Location : GCPT-8　Cone Used : 347TC　Job No. : 00086-045-003



Depth Increment : .05 m　Max Depth : 28.53 ft

Dames & Moore

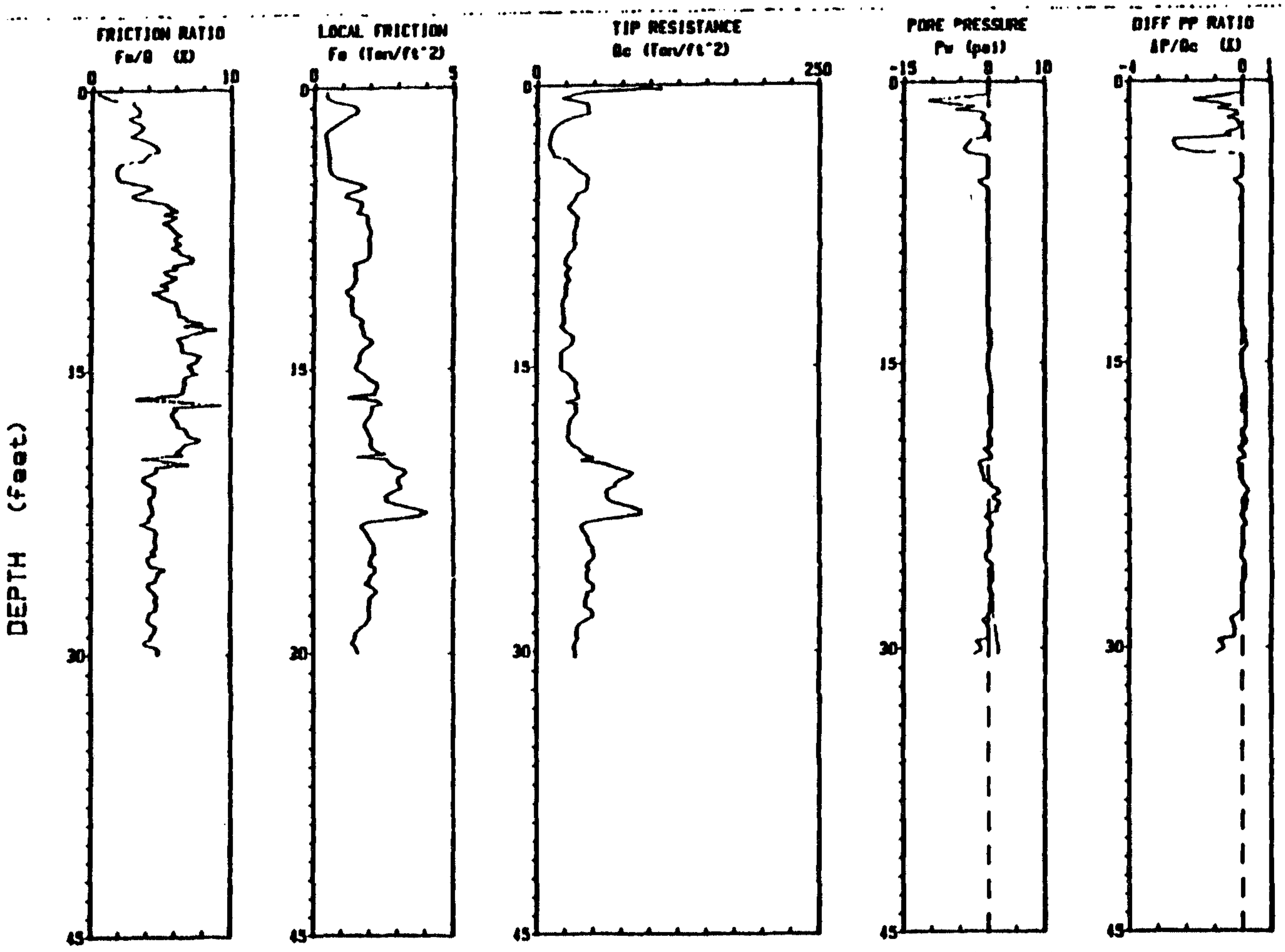
A-12

V B I

Operator : CARL G. MORGAN
Location : GCPT-10

CPT Date : 02-28-92 09:15
Cone Used : 347TC

Sounding : 920075 Pg 1 / 1
Job No. : 0088-045-003



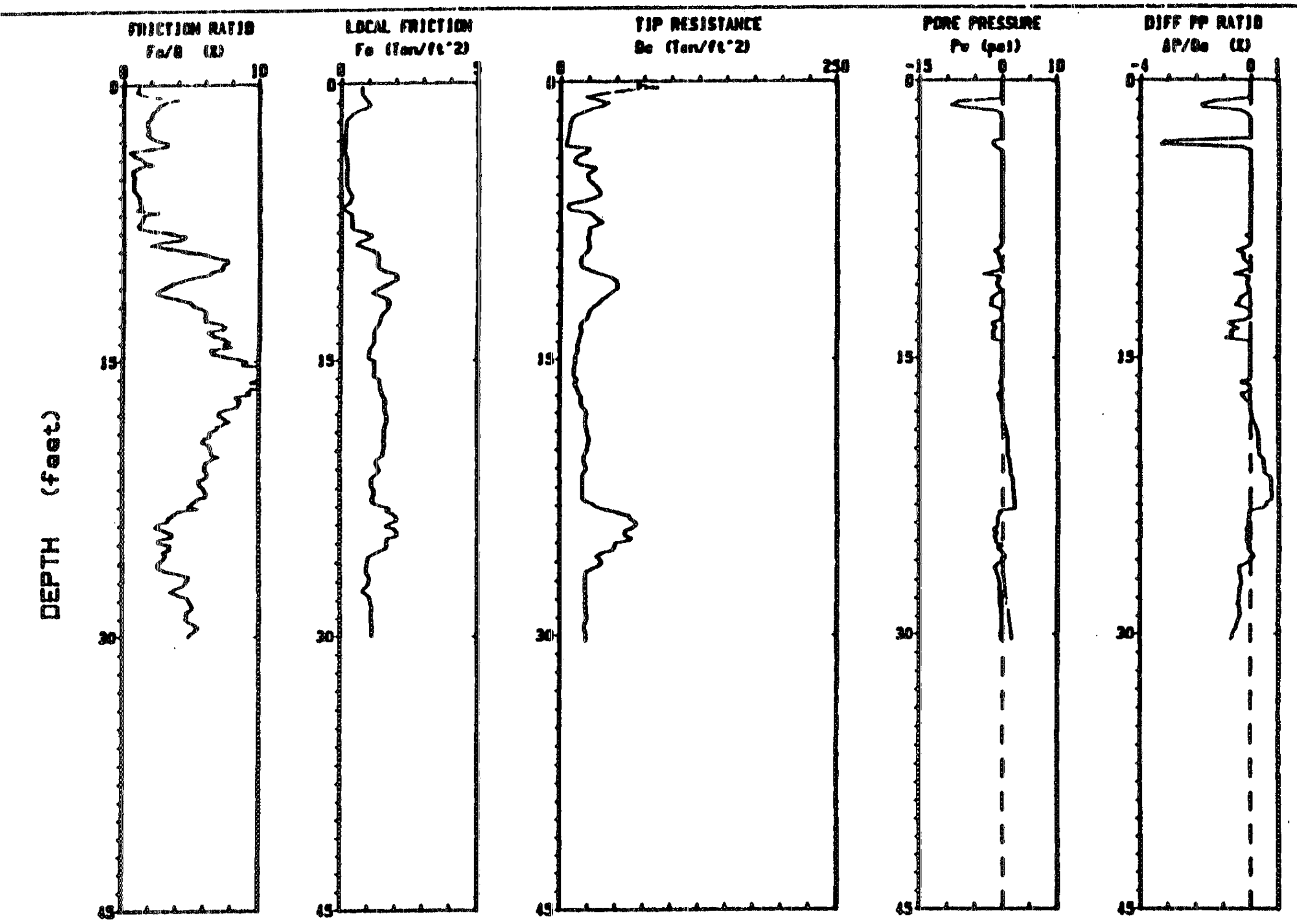
Depth Increment : .05 m

Max Depth : 30.35 ft

CHECKED BY ______ FILE 00086-045-005 ______ BY ______ DATE ______

V B I

Operator : CARL G. MORGAN CPT Date : 02-27-82 13:54 Sounding : 820074 Pg 1 / 1
Location : GCPT-11 Cone Used : 347TC Job No. : -003



Depth Increment : .05 m Max Depth : 30.35 ft

Dames & Moore

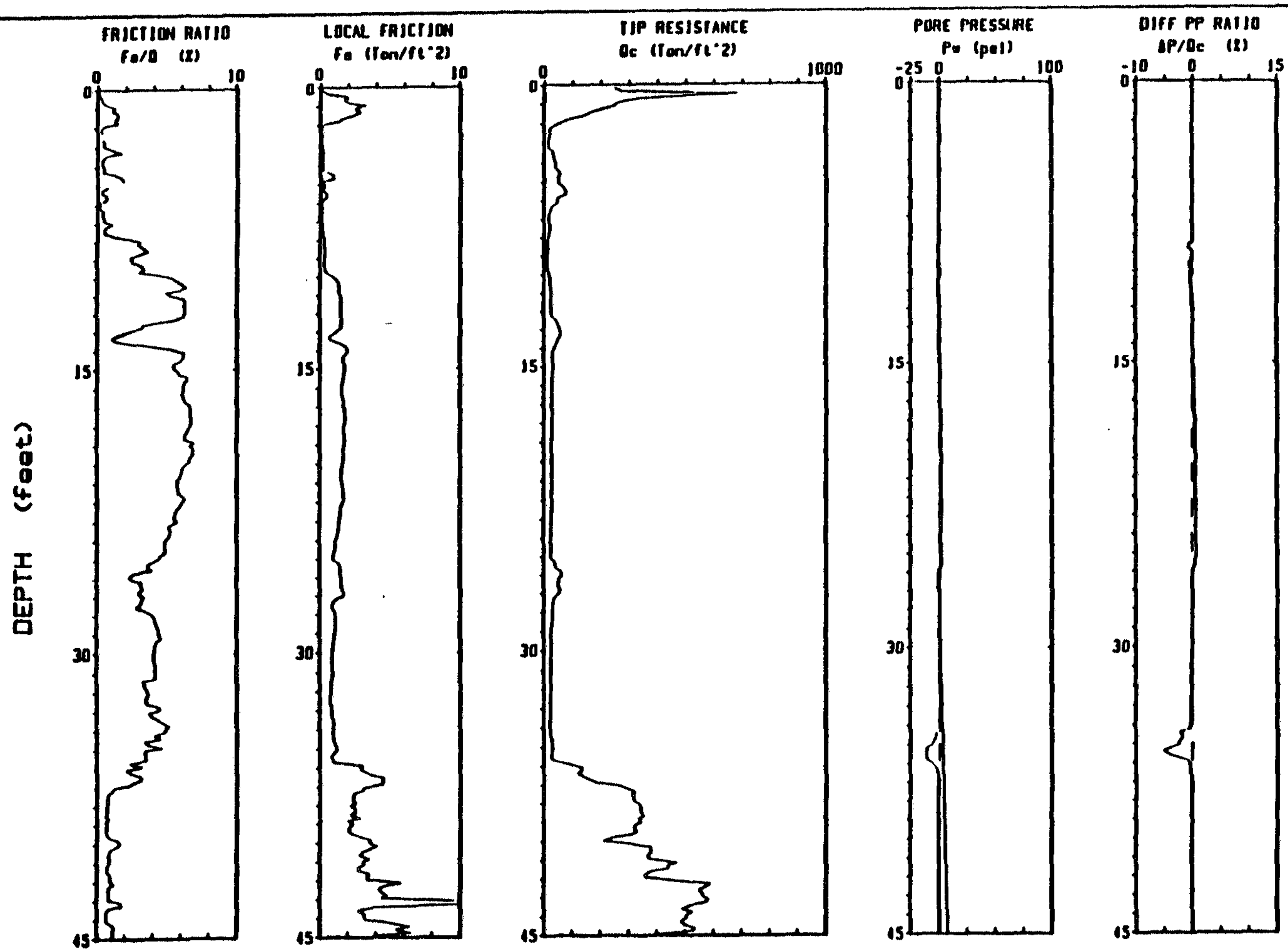
BY______DATE______

V B I

Operator : CARL G. MORGAN
Location : GCPT-12

CPT Date : 03-10-92 12:21
Cone Used : 347TC

Sounding : 920110 Pg 1 / 2
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 83.50 ft

Dames & Moore

A-15a

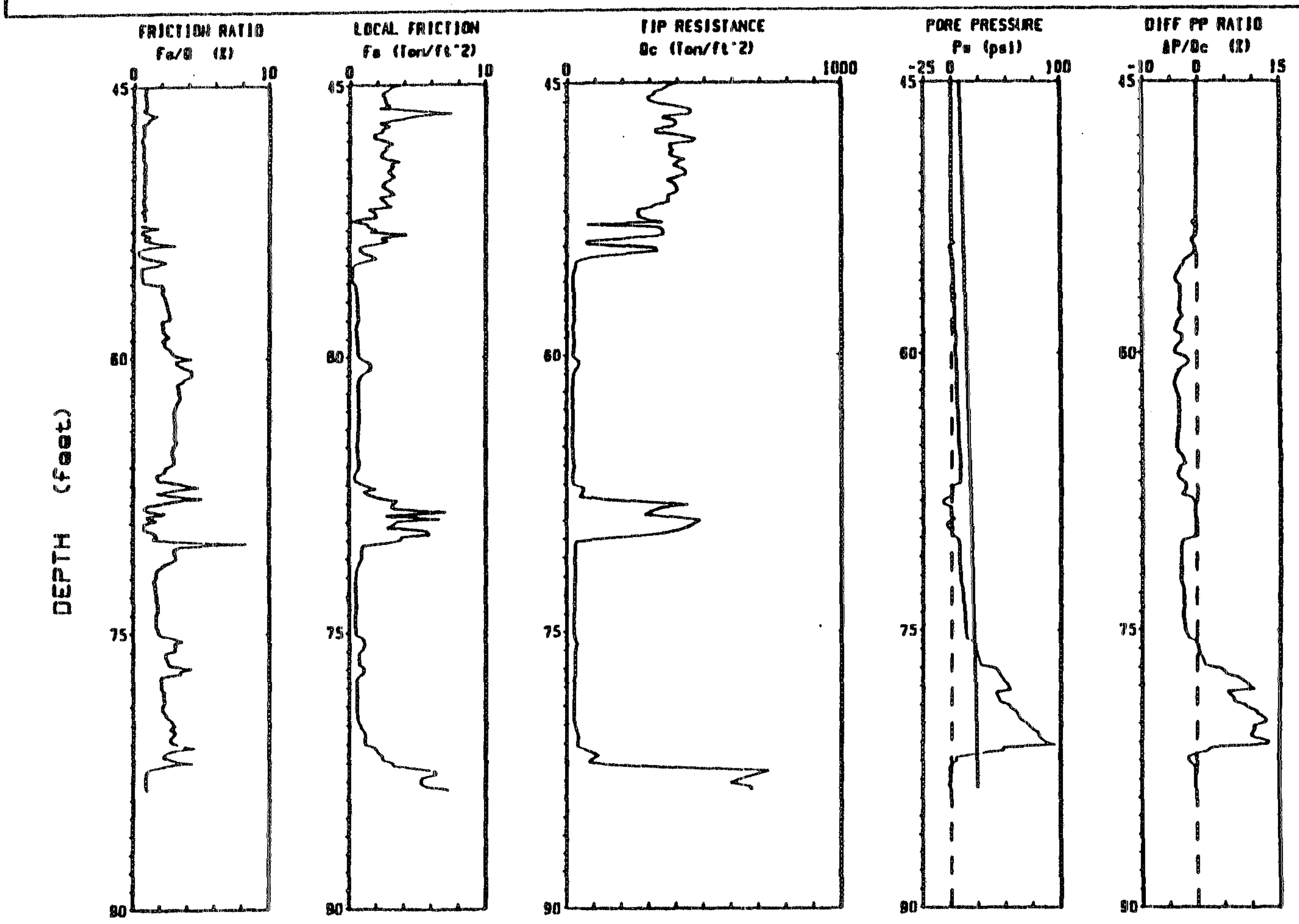
CHECKED BY ____________ FILE 00086-045-003 ____________ BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-12

CPT Date : 03-10-92 12:21
Cone Used : 347TC

Sounding : 920110 Pg 2 / 2
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 83.50 ft

Dames & Moore

A-16b

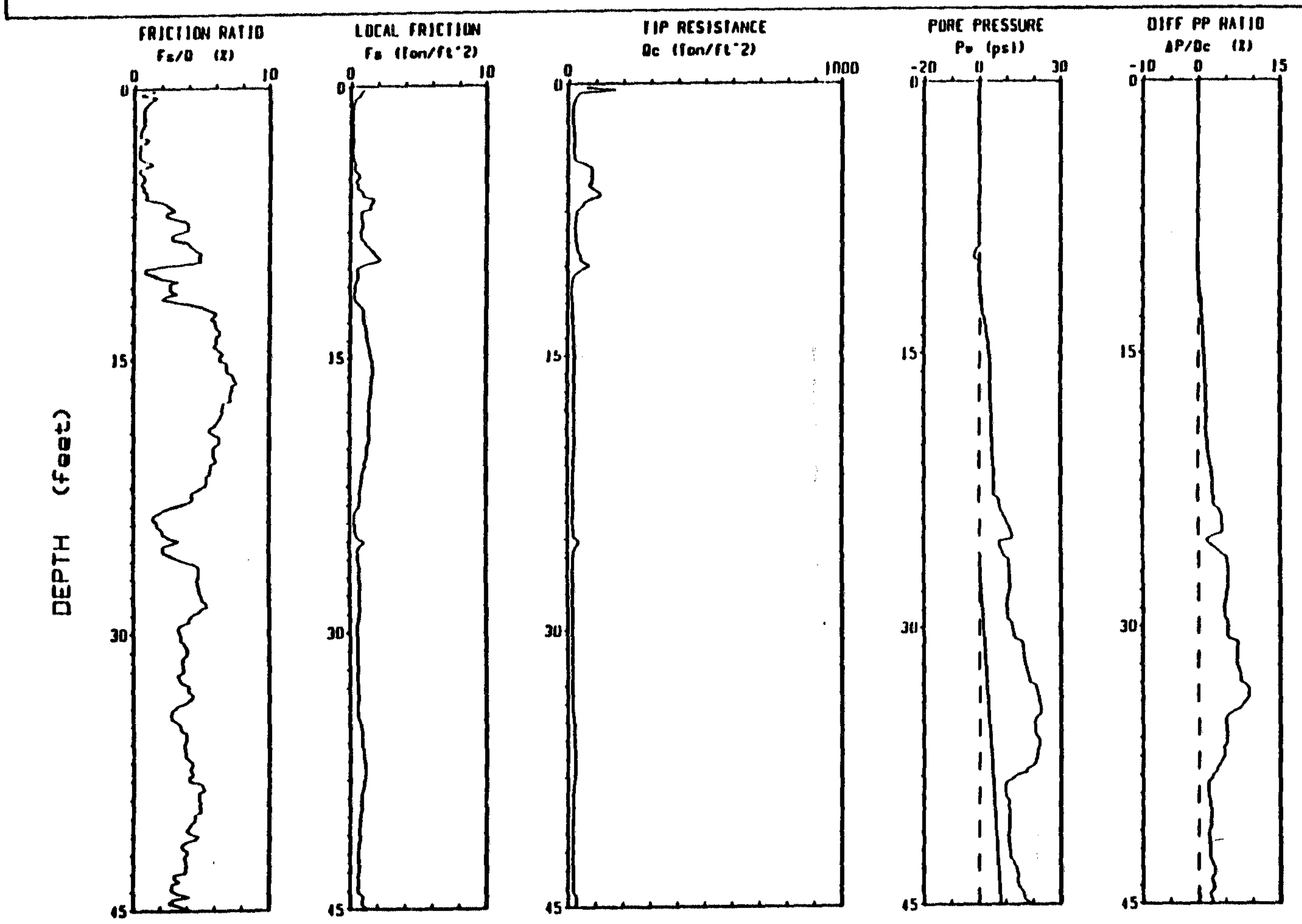
BY________ DATE______

V B I

Operator : CARL G. MORGAN
Location : GCPT-13

CPT Date : 03-10-92 09:30
Cone Used : 347TC

Sounding : 920109 Pg 1 / 2
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 67.91 ft

Dames & Moore

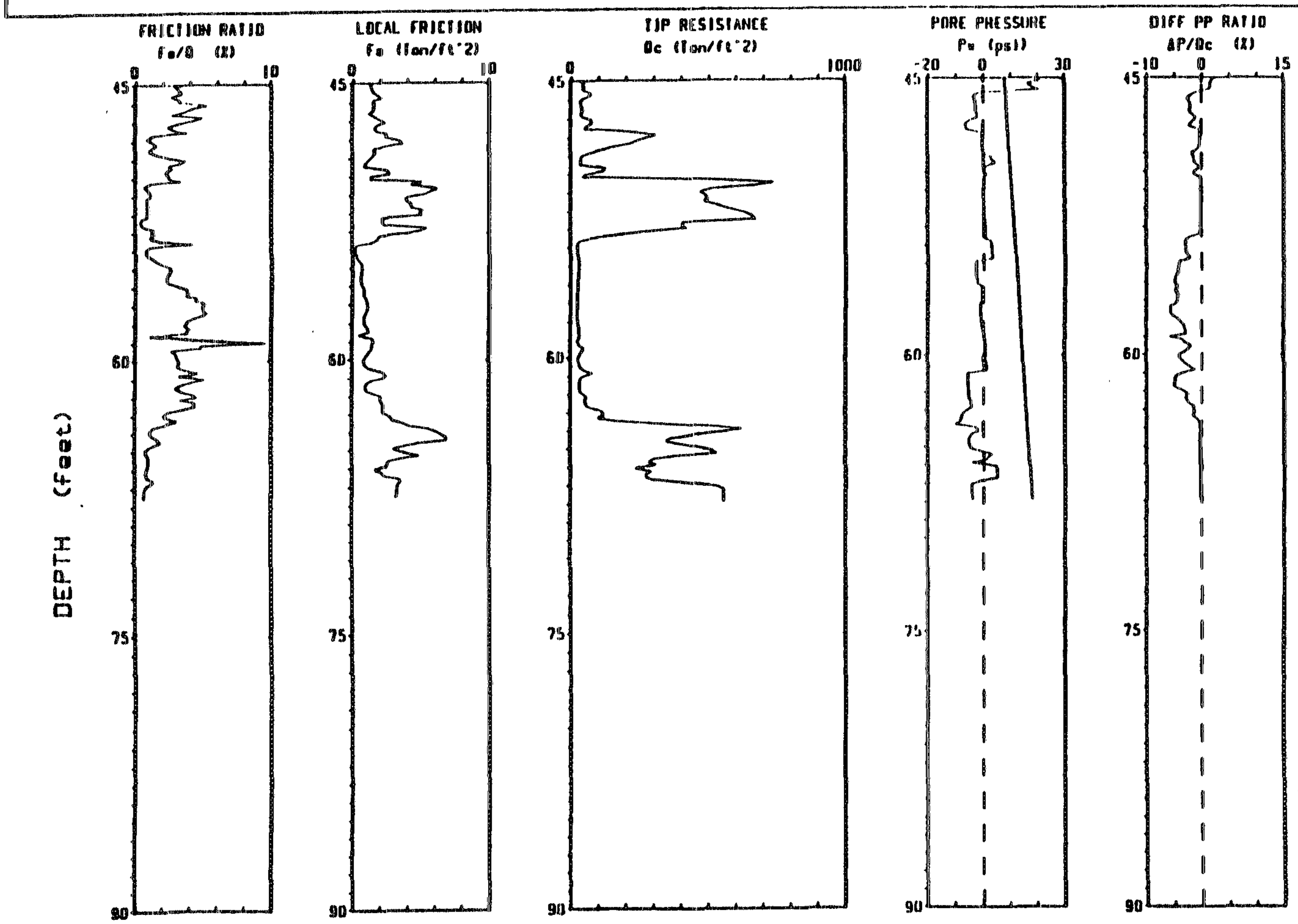
A-16a

BY ______ DATE ______
CHECKED BY ______
FILE 00086-045-003

REVISIONS
BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-13
CPT Date : 03-10-92 09:38
Cone Used : 347TC
Sounding : 920109 Pg 2 / 2
Job No. : 0086-045-003



Depth Increment : .05 m
Max Depth : 67.91 ft

Dames & Moore

A-166

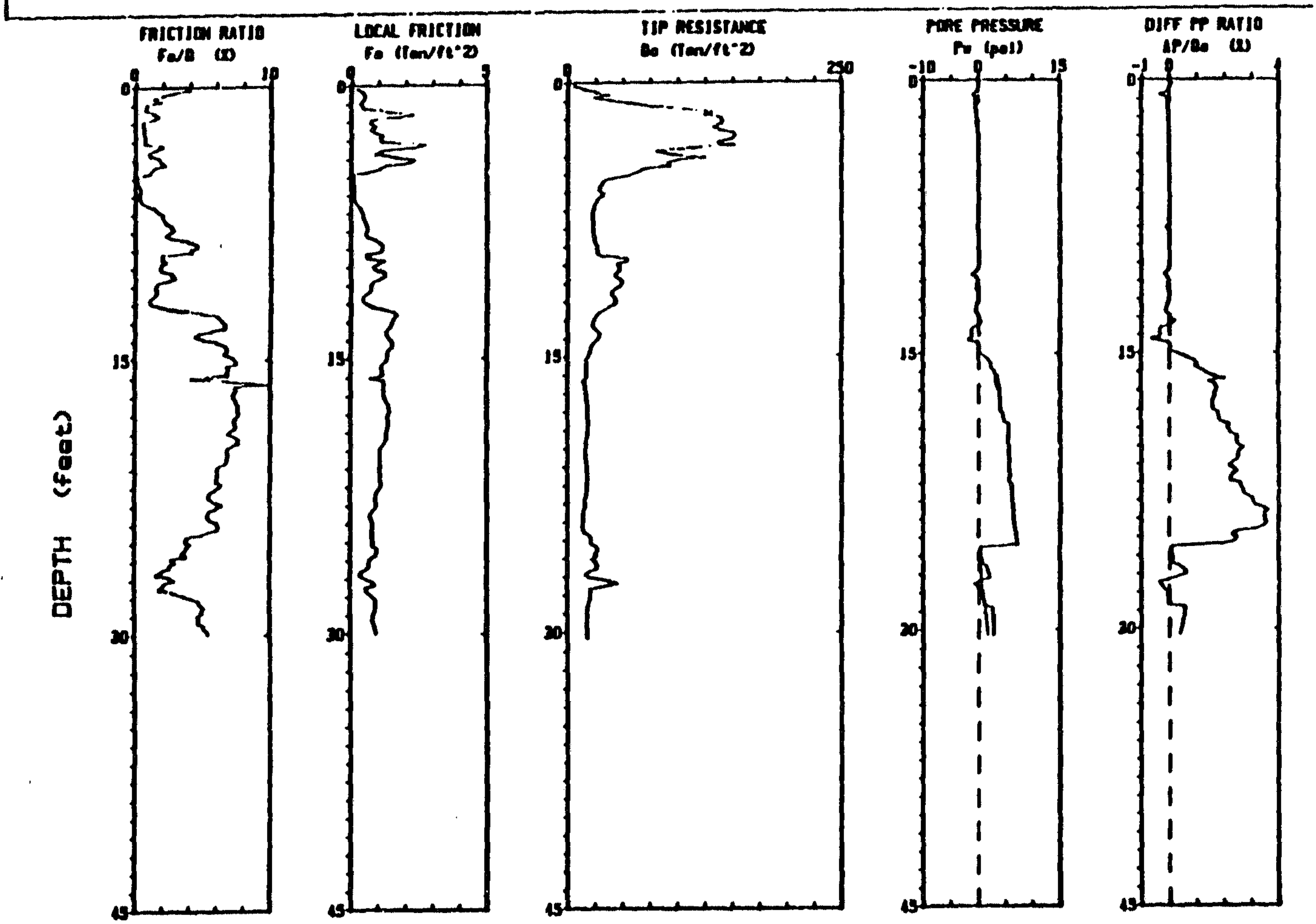
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : CCPT-14

CPT Date : 02-28-92 10:37
Cone Used : 347TC

Sounding : 920078 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

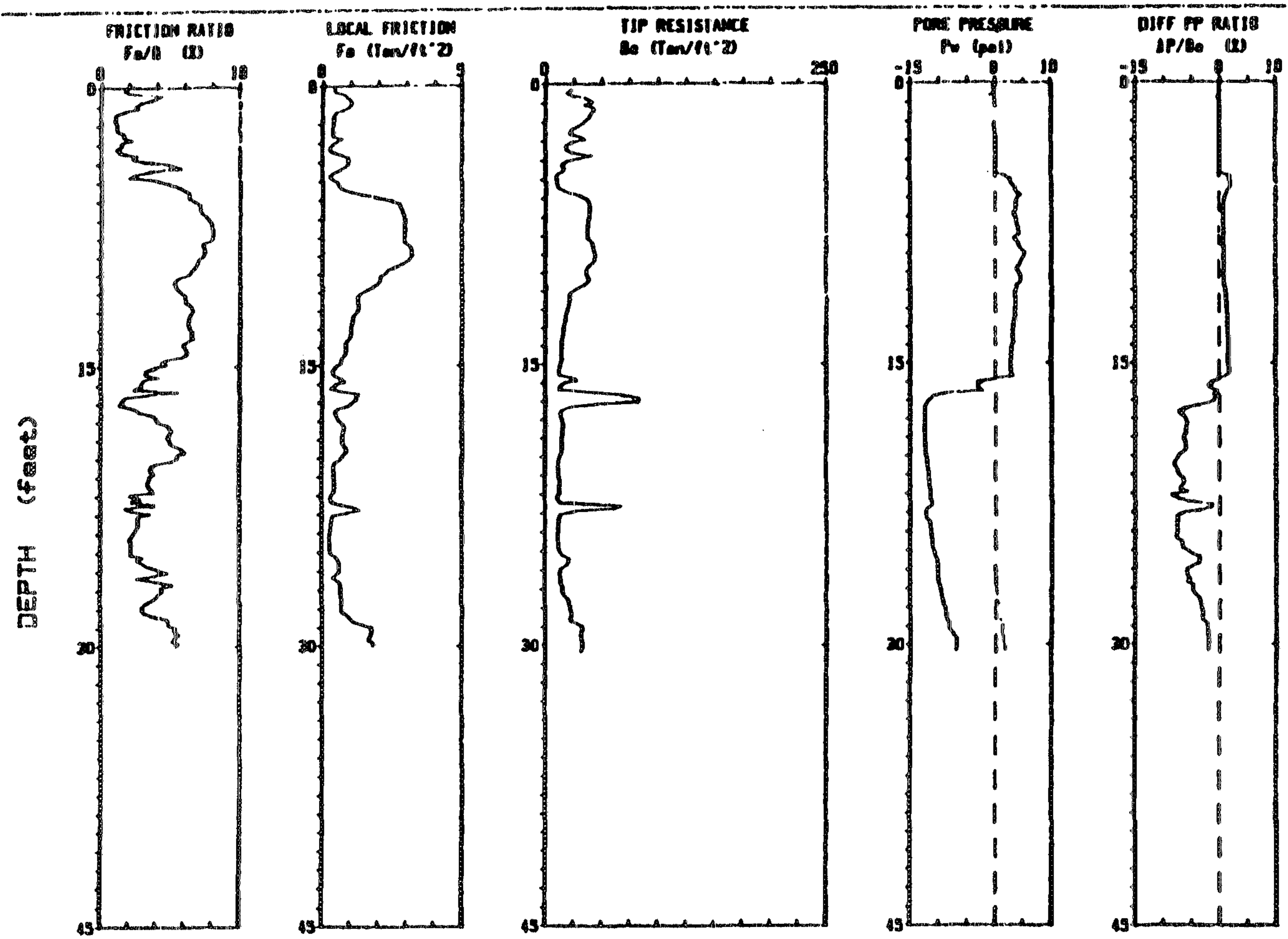
A-17

V B I

Operator : CARL G. MORGAN
Location : GCPT-18

CPT Date : 02-28-92 12:10
Cone Used : 347TC

Sounding : 020077 Pg 1 / 1
Job No. : 0088-045-003



Depth Increment : .05 m

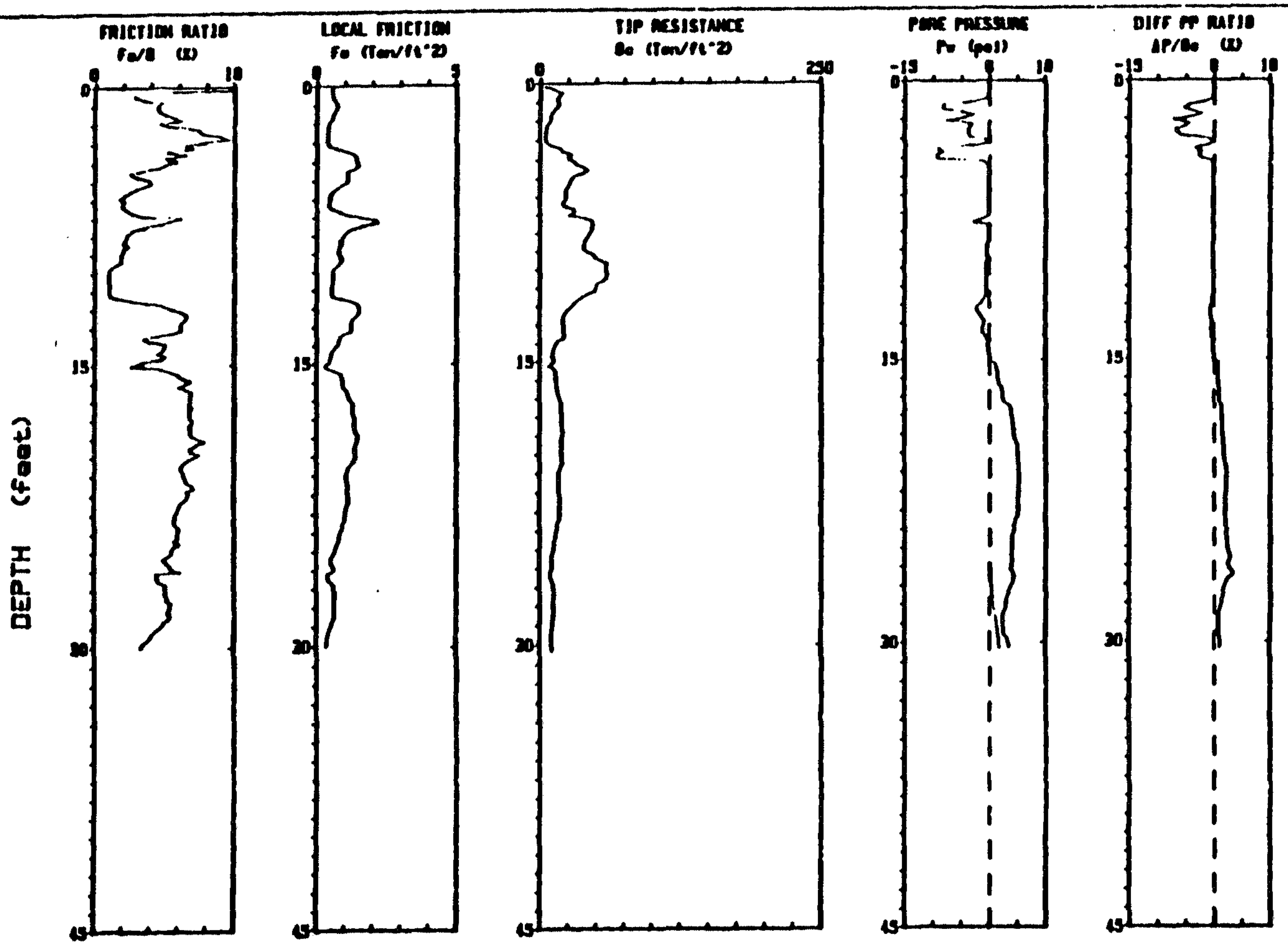
Max Depth : 30.35 ft

V B I

Operator : CARL G. MORGAN
Location : GCPT-17

CPT Date : 02-28-82 13:18
Cone Used : 347TC

Sounding : 920078 Pg 1 / 1
Job No. : 0066-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-19

BY ________ DATE ________
CHECKED BY ________

FILE 00086-045-003

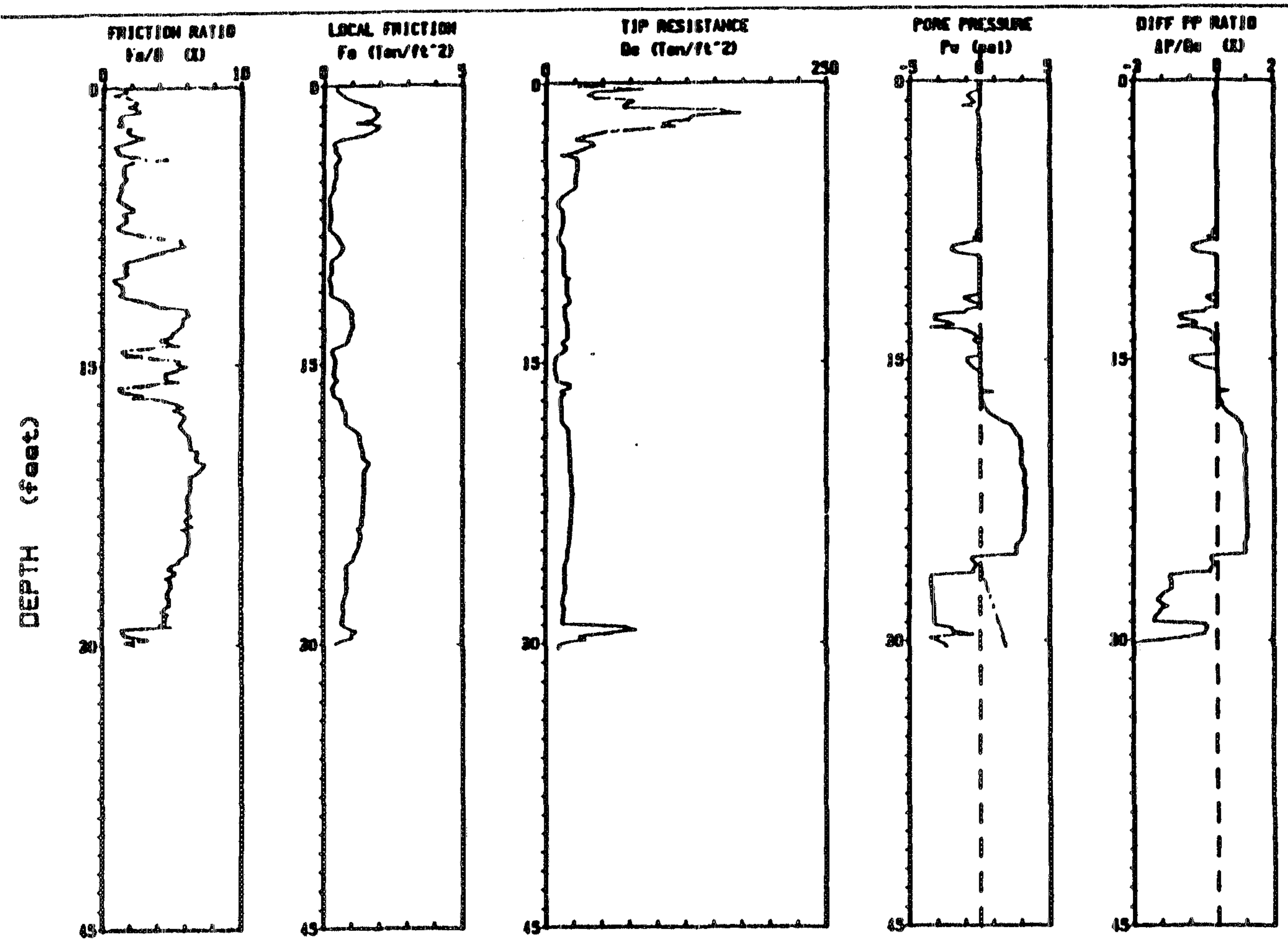
REVISIONS
BY ________ DATE ________

V B I

Operator : CARL G. MORGAN
Location : GCPT-18

CPT Date : 02-28-92 14:38
Cone Used : 347TC

Sounding : 920079 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

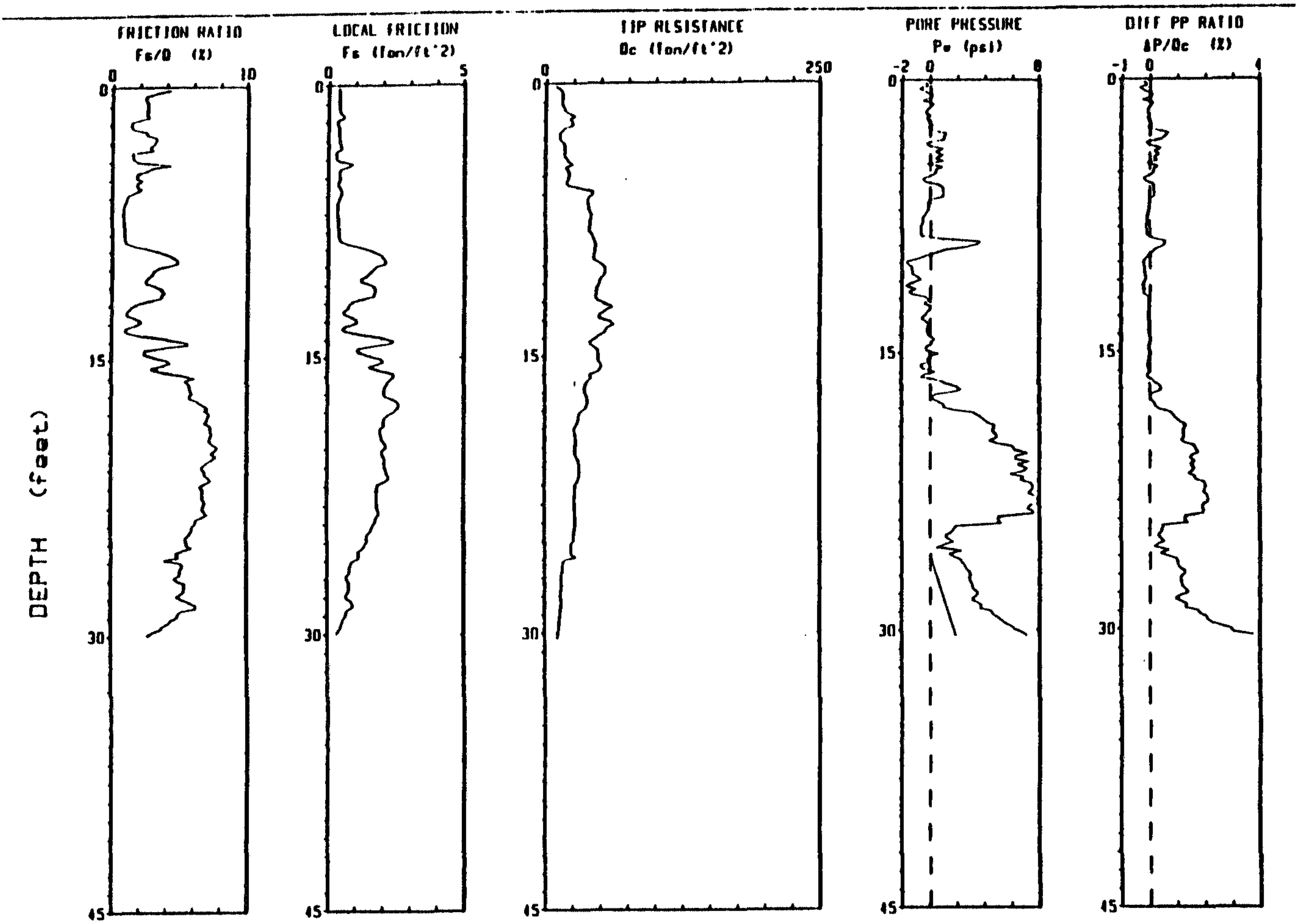
A-20

V B I

Operator : CARL G. MORGAN
Location : GCPT-19(A)

CPT Date : 03-02-92 10:24
Cone Used : 347TC

Sounding : 920081 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

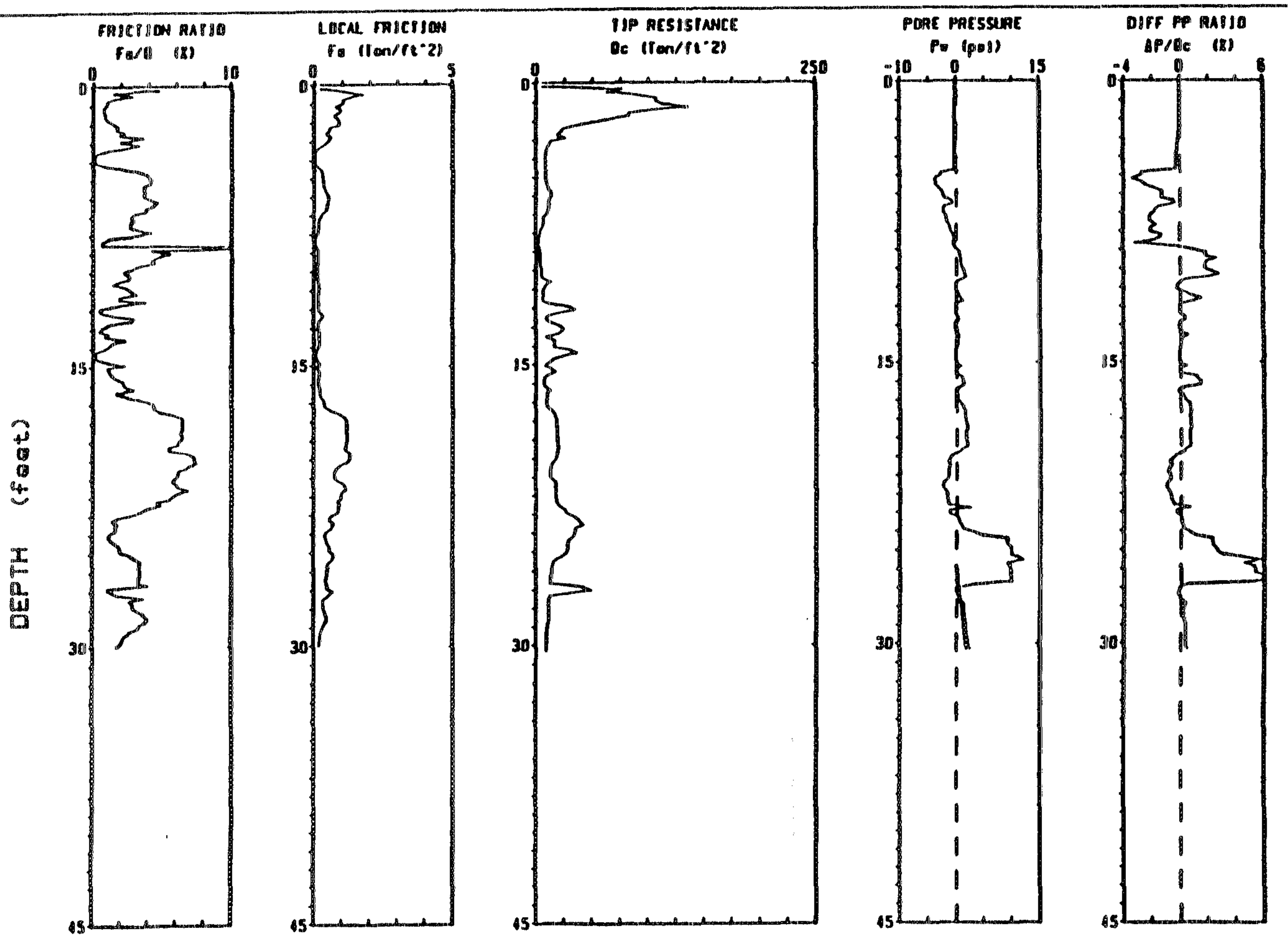
CHECKED BY ______________ FILE 00086-045-003 REVISIONS BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-20

CPT Date : 03-10-92 16:03
Cone Used : 347TC

Sounding : 920112 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m Max Depth : 30.35 ft

Dames & Moore

A-22

CHECKED BY ______ FILE 00086 045-003 ______

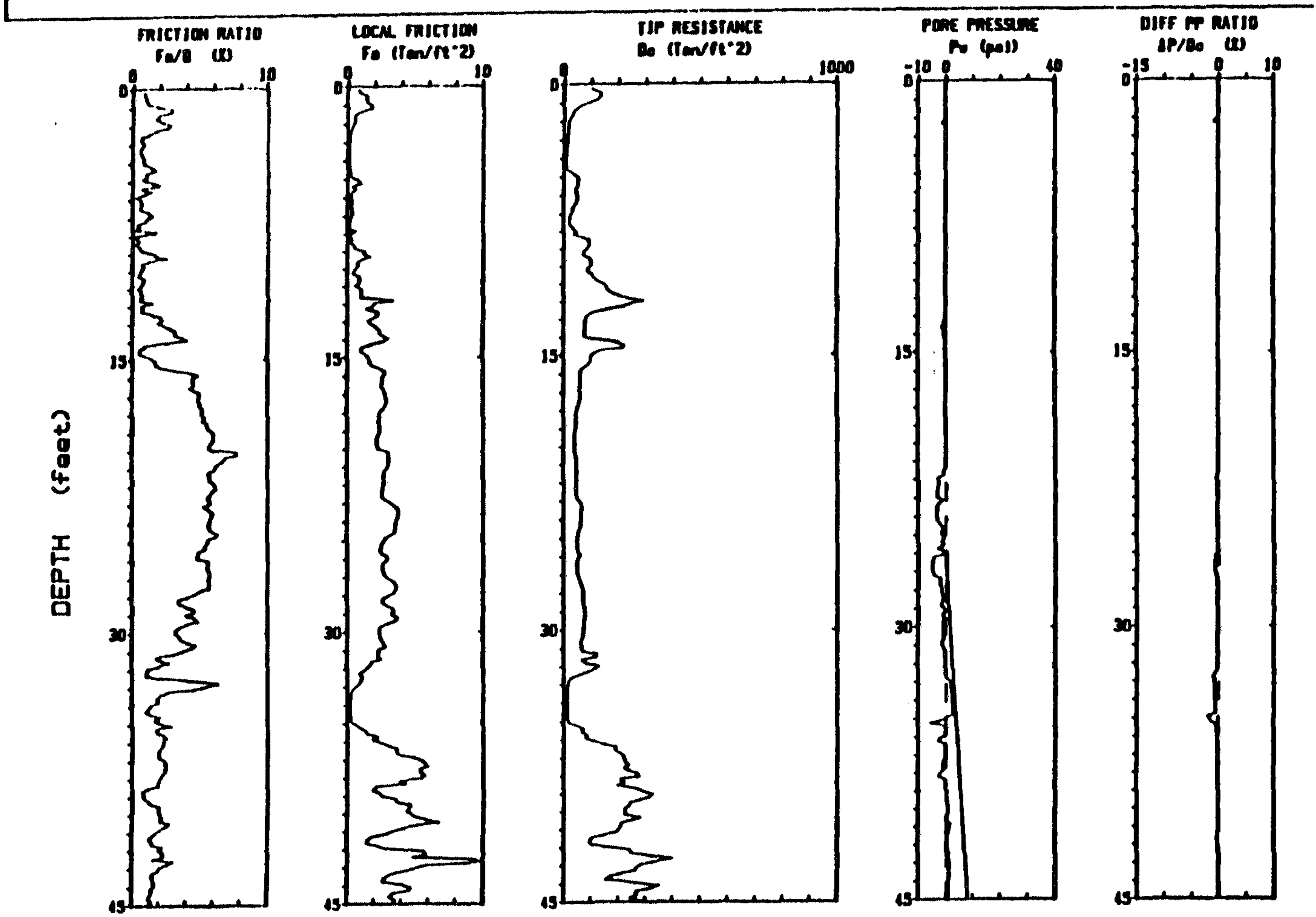
REVISIONS
BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-21

CPT Date : 03-12-82 14:22
Cone Used : 347TC

Sounding : 920115 Pg 1 / 2
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 84.48 ft

Dames & Moore

4-230

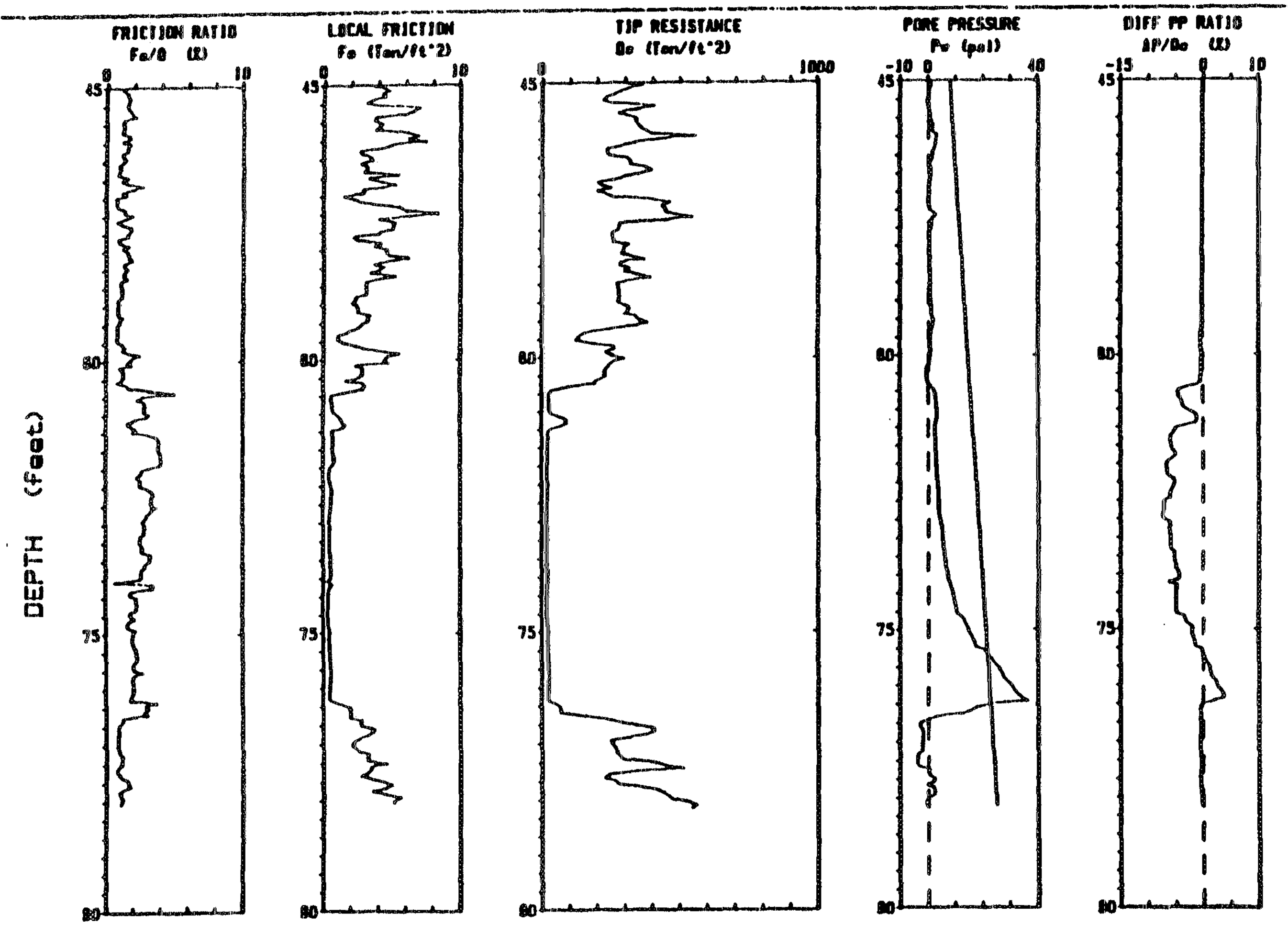
CHECKED BY ______ FILE 00086-045-003 ______ REVISIONS BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-21

CPT Date : 03-12-92 14:22
Cone Used : 347TC

Sounding : 920115 Pg 2 / 2
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 84.48 ft

Dames & Moore

REVISIONS

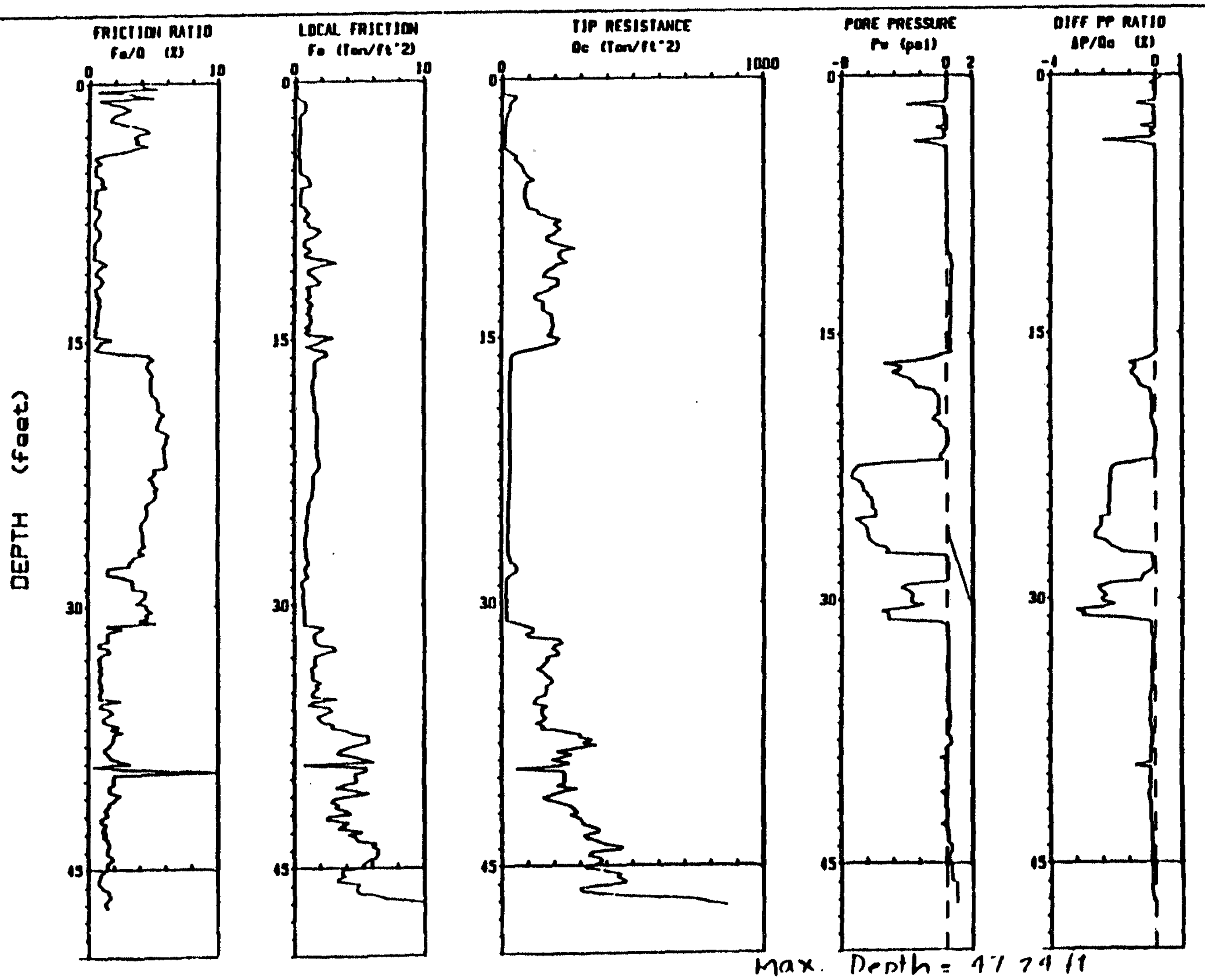
CHECKED BY ______ FILE 0008 045-003 BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-22

CPT Date : 03-16-92 14:33
Cone Used : 347TC

Sounding : 920116 Pg 1
Job No. : 0088-045-003



Max. Depth = 47.79 ft

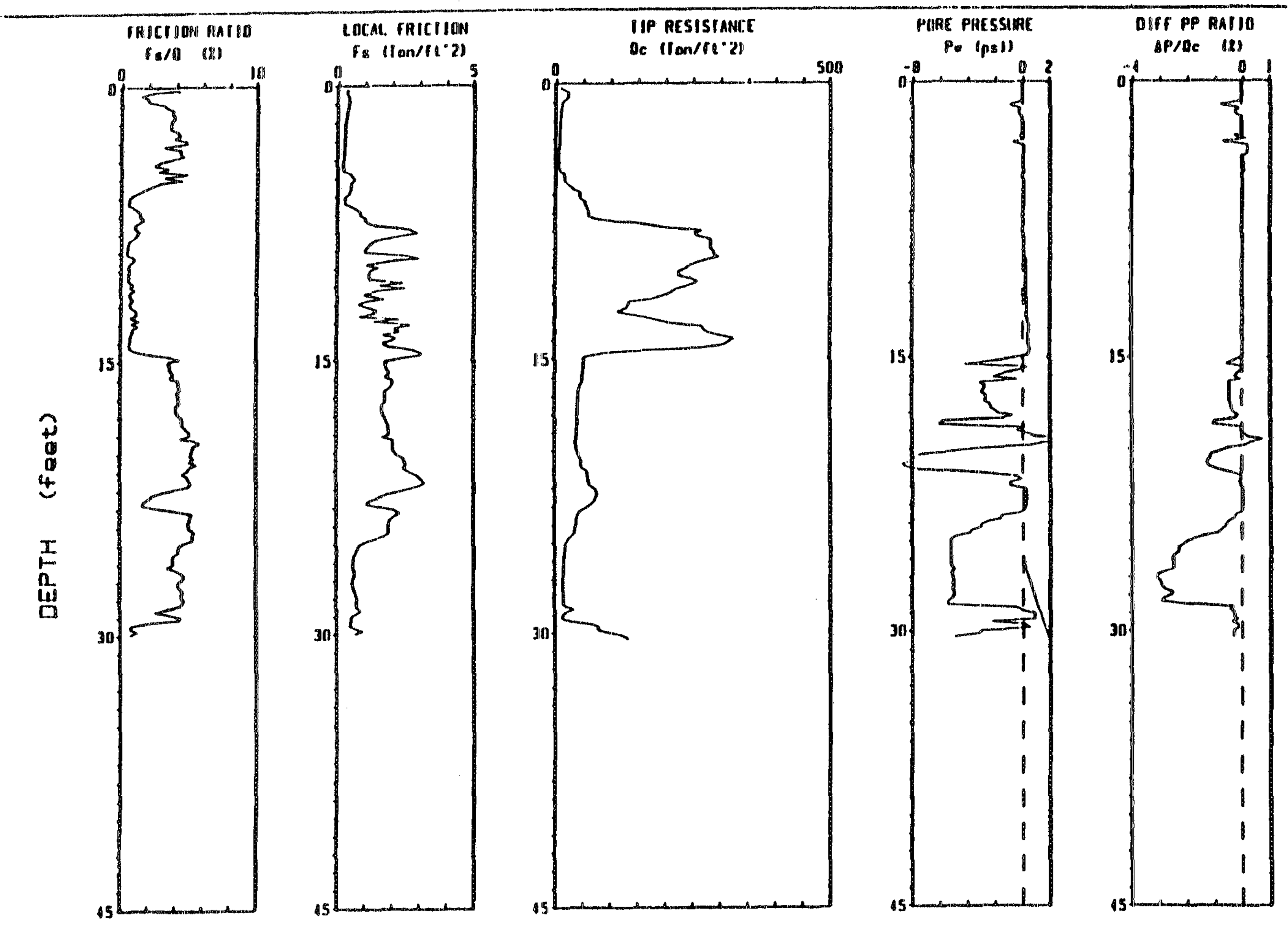
Dames & Moore

A-74

CHECKED BY ______ FILE 00086-045-003 REVISIONS BY ______ DATE ______

V B I

Operator : CARL G. MORGAN CPT Date : 03-02-92 11:35 Sounding : 920082 Pg 1 / 1
Location : GCPT-23 Cone Used : 347TC Job No. : 0086-045-003



Depth Increment : .05 m Max Depth : 30.35 ft

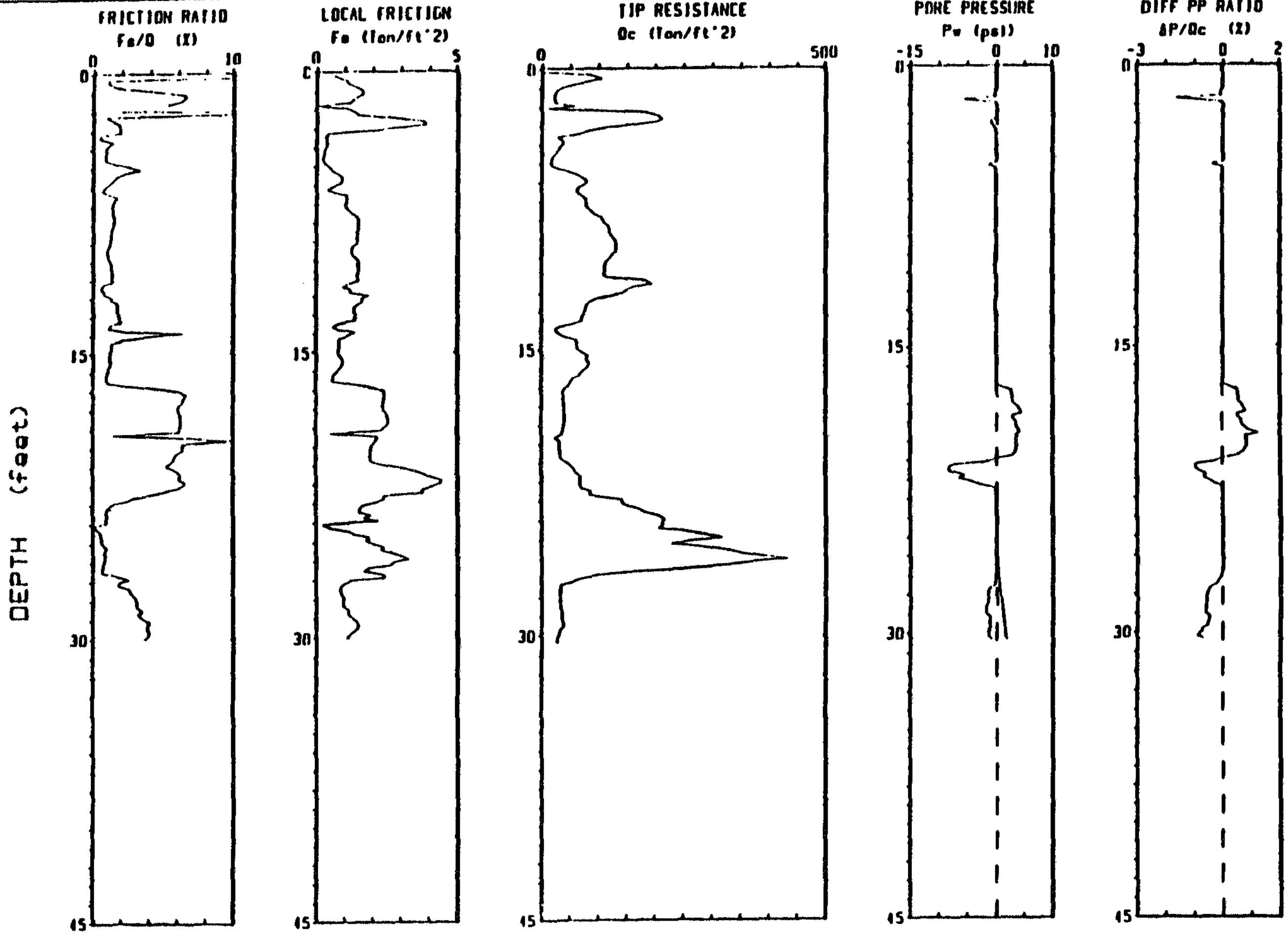
CHECKED BY ____________ FILE 00086- 45-003 ____________ REVISIONS BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-24

CPT Date : 03-02-92 12:45
Cone Used : 347TC

Sounding : 920083 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

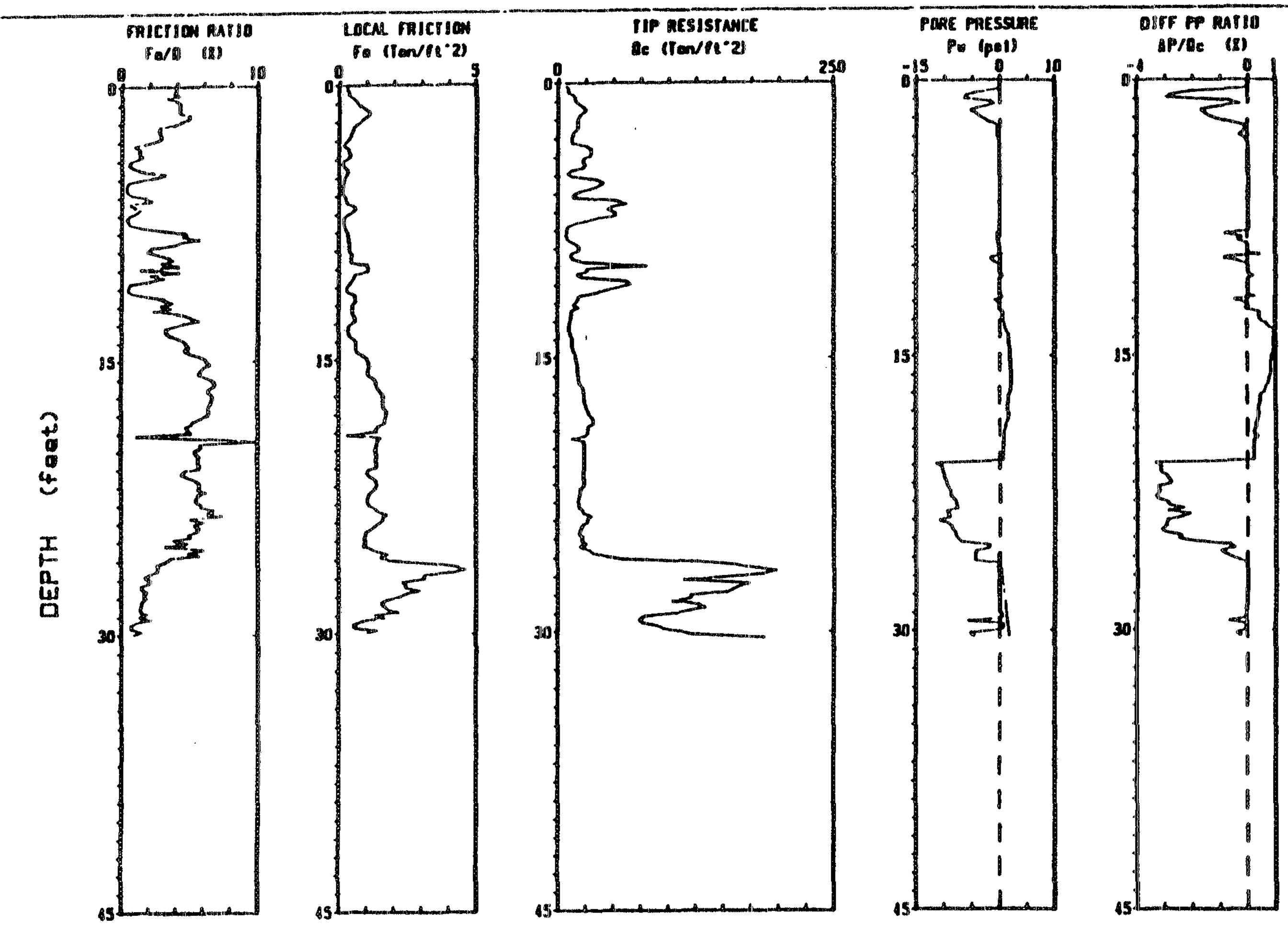
Dames & Moore

CHECKED BY ______________ FILE 00086-045-003 BY ________ DATE ______

V B I

Operator : CARL G. MORGAN CPT Date : 02-26-92 11:17 Sounding : 920067 Pg 1 / 1

Location : GCPT-25 Cone Used : 347TC Job No. : 00086-045-003



Depth Increment : .05 m Max Depth : 30.35 ft

Dames & Moore

A-67

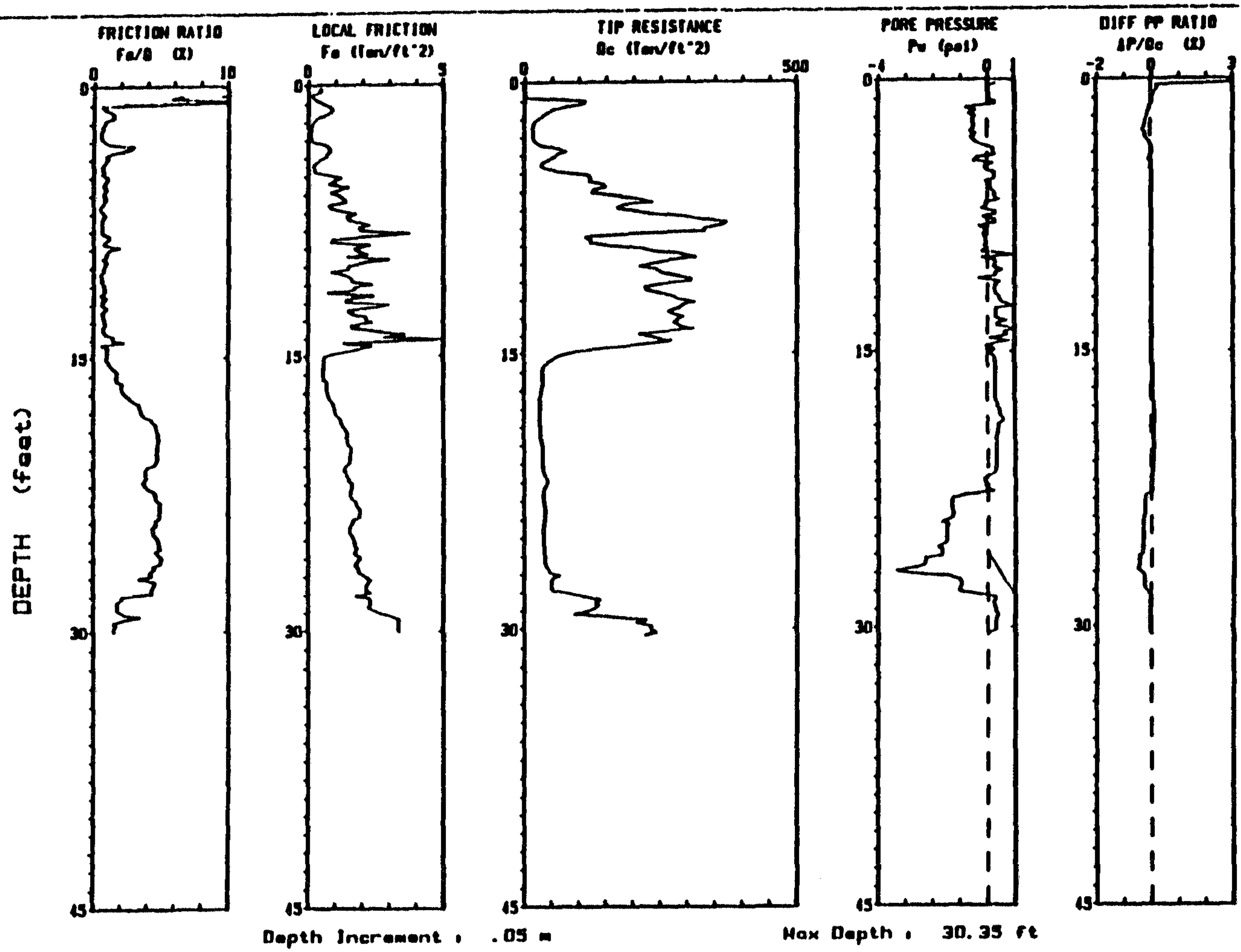
CHECKED BY ______ FILE 00086-[illegible]45-003 BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-28

CPT Date : 02-28-92 12:45
Cone Used : 347TC

Sounding : 920088 Pg 1 / 1
Job No. : 00086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-28

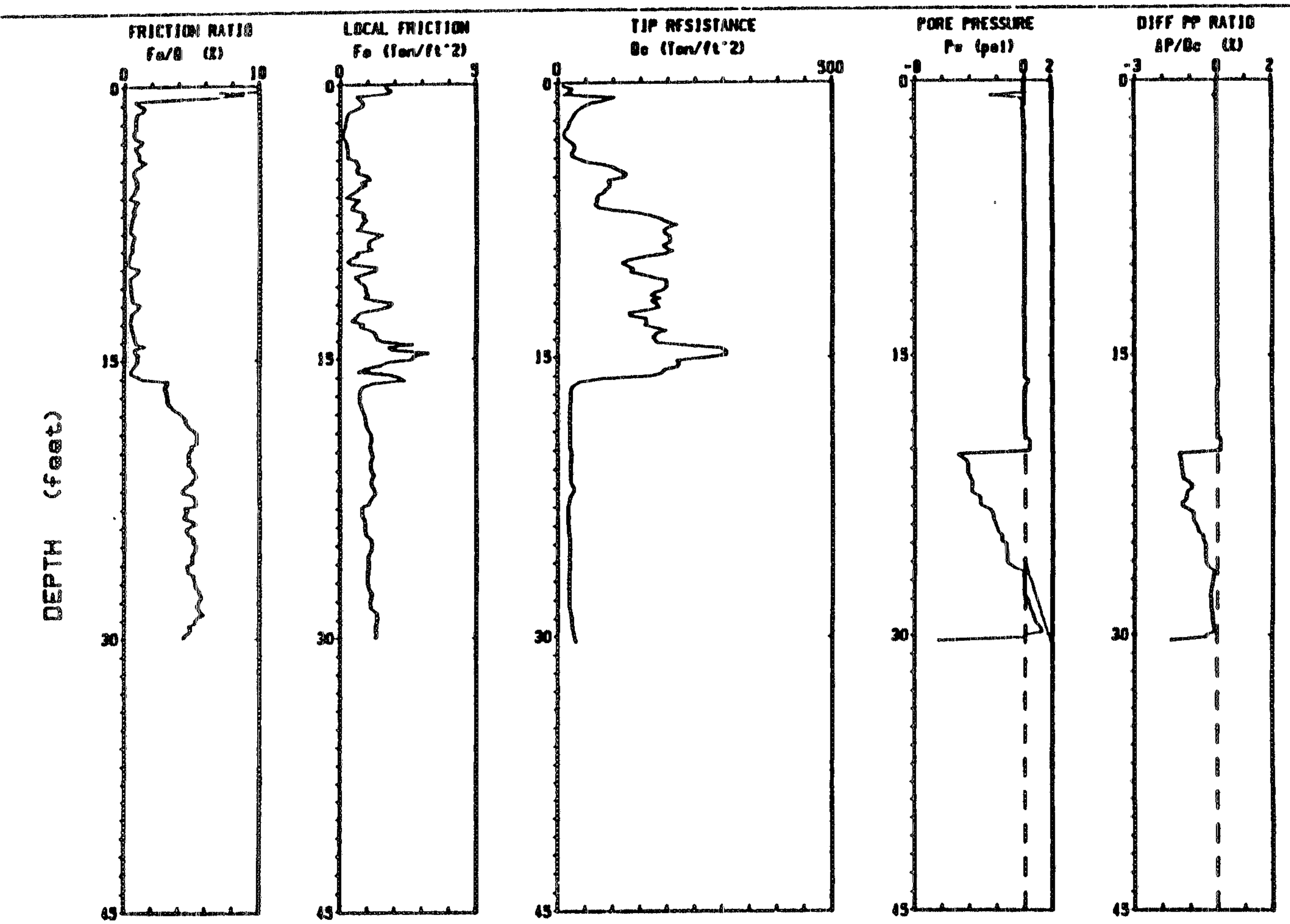
CHECKED BY ____________ FILE 00086-045-003 BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-27

CPT Date : 02-26-92 14:17
Cone Used : 347TC

Sounding : 920069 Pg 1 / 1
Job No. : 00086-045-003



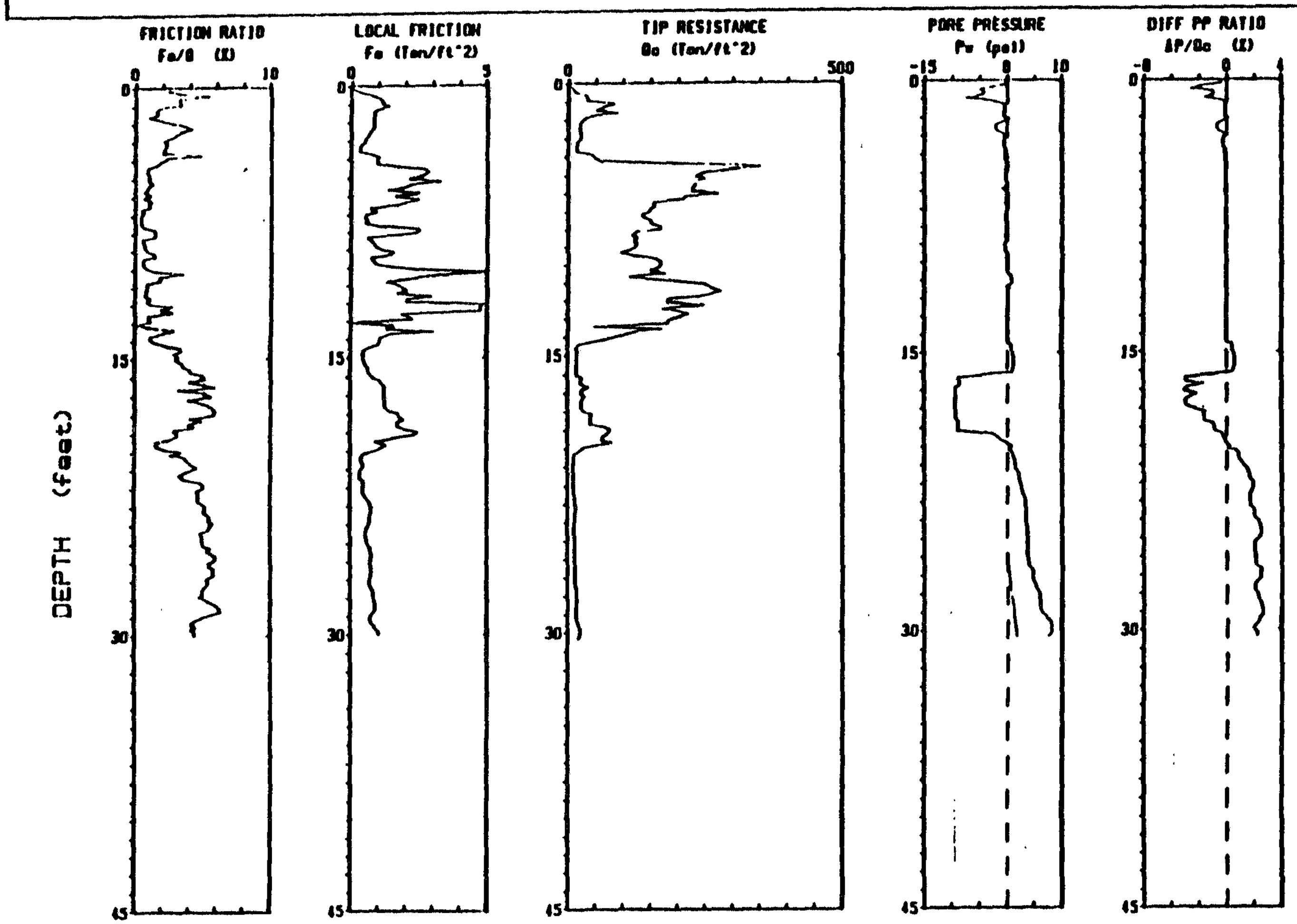
Depth Increment : .05 m

Max Depth : 30.35 ft

CHECKED BY ____________ FILE 00086 ?45 - 003 ____________ BY ________ DATE ______

V B I

Operator : CARL G. MORGAN CPT Date : 03-18-82 17:38 Sounding : 820117 Pg 1 / 1
Location : GCPT-28 Cone Used : 347TC Job No. : 0086-045-003



Depth Increment : .05 m Max Depth : 30.35 ft

Dames & Moore

BY ____________ DATE ____________

CHECKED BY ______________

FILE 00086-045-003

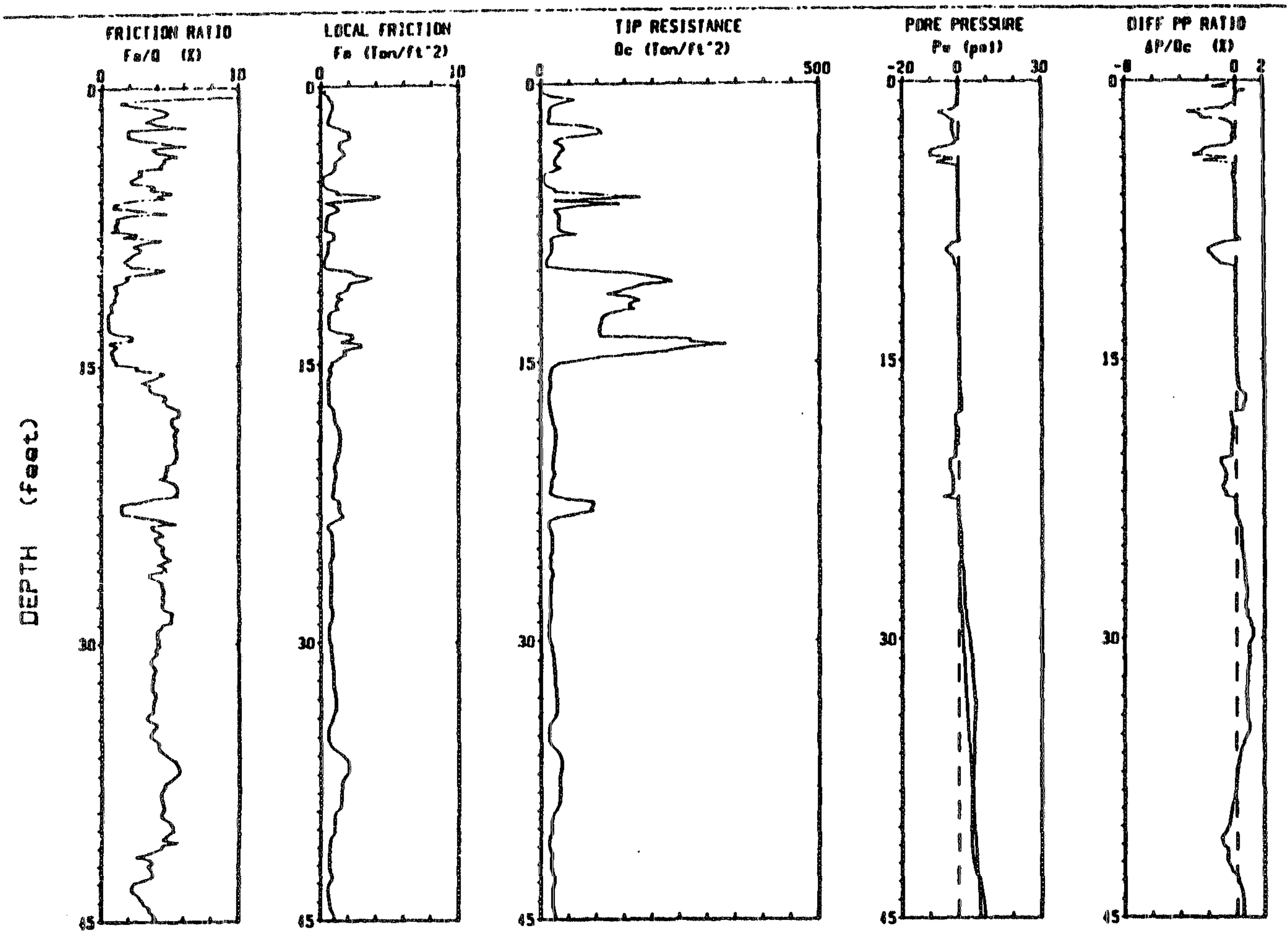
REVISIONS

BY __________ DATE ______

V B I

Operator : CARL G. MORGAN   CPT Date : 03-05-92 13:56   Sounding : 920088 Pg 1 / 2

Location : GCPT-28   Cone Used : 347TC   Job No. : 0086-045-003



Depth Increment : .05 m   Max Depth : 71.85 ft

Dames & Moore

A-31C

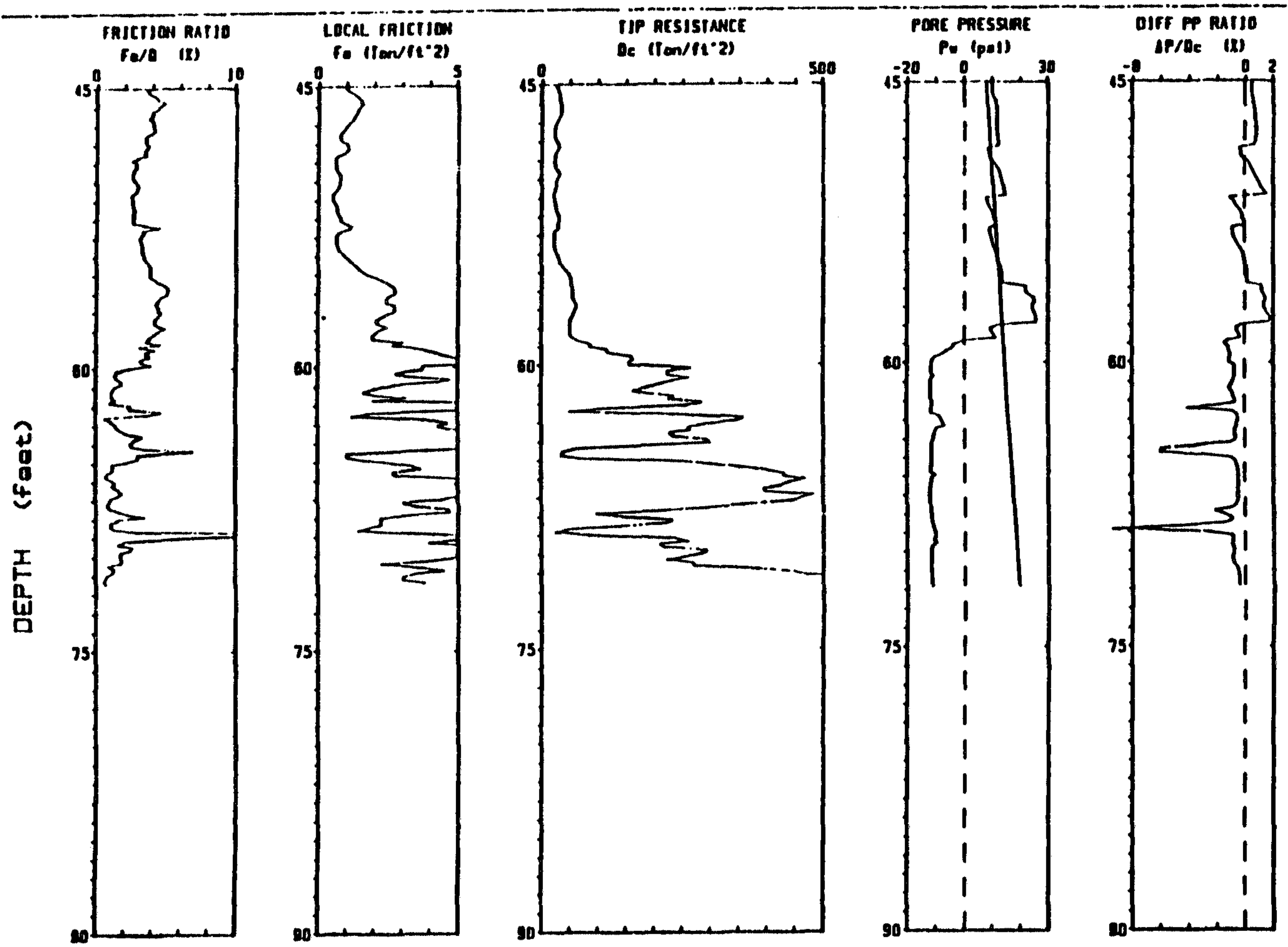
CHECKED BY ________ FILE 00086 045-003 ________ BY ________ DATE ________

V B I

Operator : CARL G. MORGAN
Location : GCPT-29

CPT Date : 03-05-82 13:56
Cone Used : 347TC

Sounding : 920089 Pg 2 / 2
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 71.85 ft

Dames & Moore

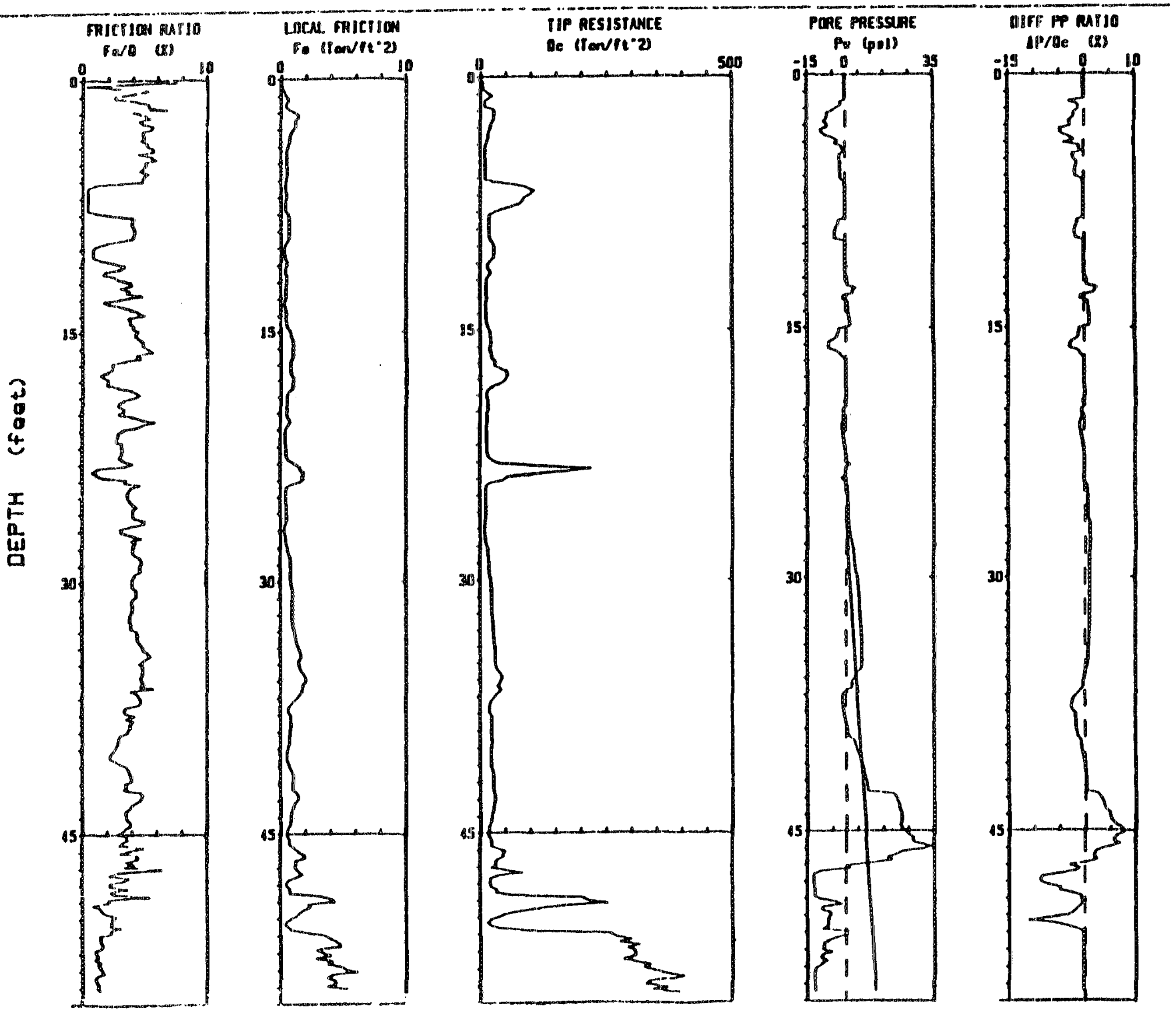
A-316

V B I

Operator : CARL G. MORGAN
Location : GCPT-30

CPT Date : 03-16-92 09:18
Cone Used : 347TC

Sounding : 920118 Pg 1
Job No. : 0086-045-003



max Depth = 54.46 ft

REVISIONS

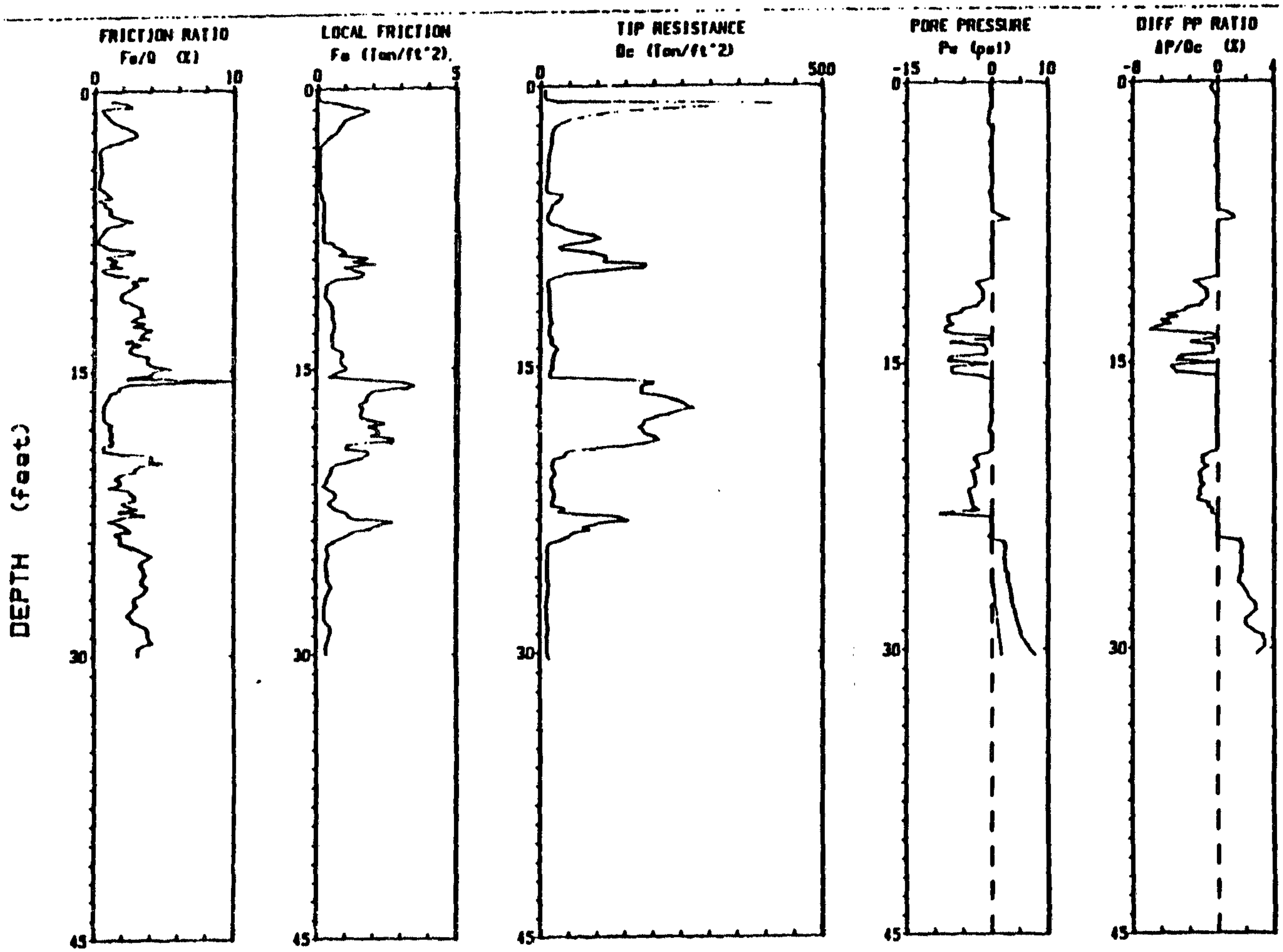
CHECKED BY ____________ FILE 0008(045-003 ____________ BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-31

CPT Date : 03-08-92 16:31
Cone Used : 347TC

Sounding : 920108 Pg 1 / 1
Job No. : 0088-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

BY ______ DATE ______
CHECKED BY ______

FILE 00086- 045-003

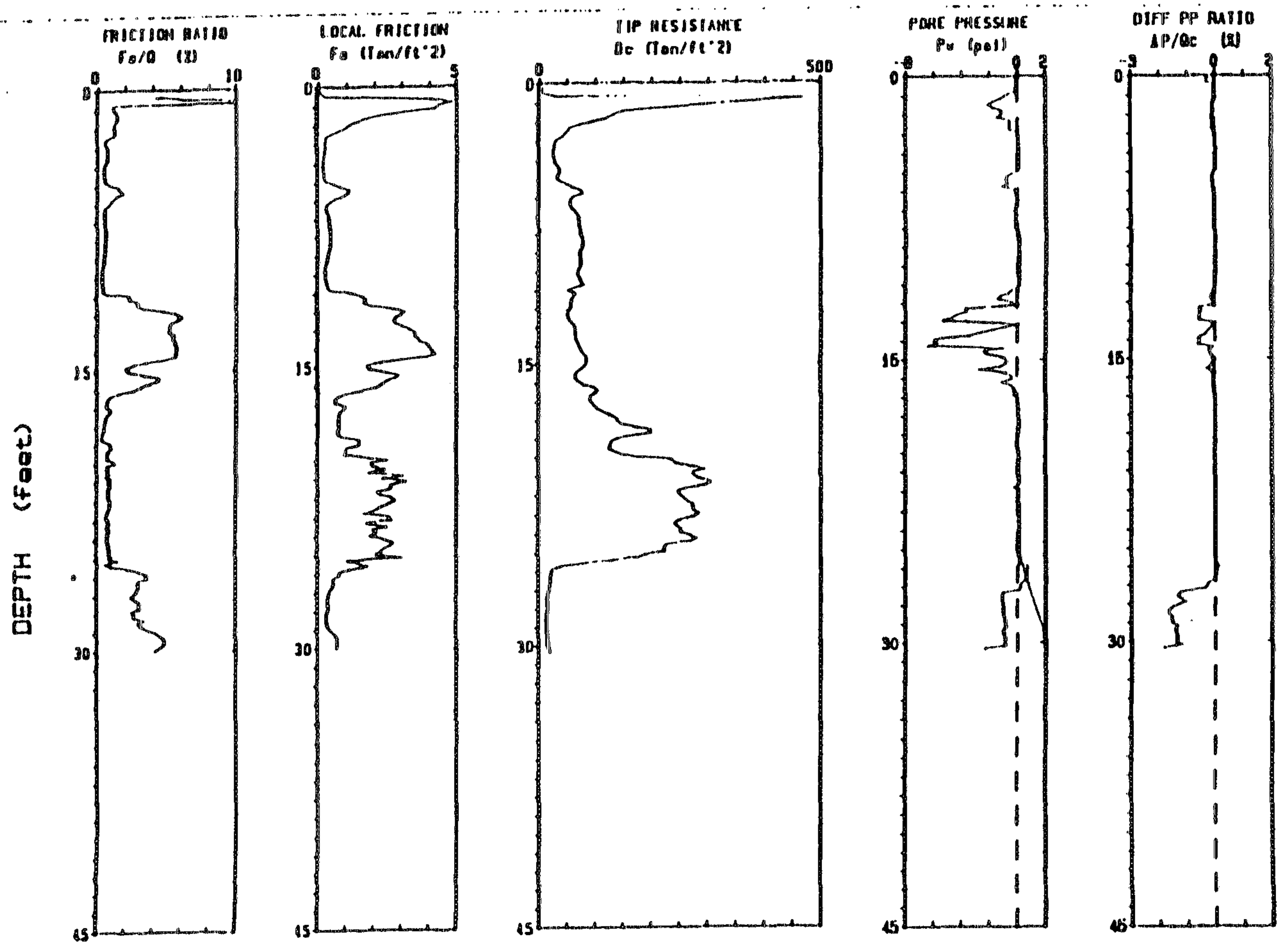
REVISIONS
BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-32

CPT Date : 03-08-82 15:05
Cone Used : 347TC

Sounding : 820107 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

BY ____________ DATE ____________

CHECKED BY ____________

FILE 00086-045-003

REVISIONS

BY ________ DATE ______

B I

Operator : CARL G. MORGAN

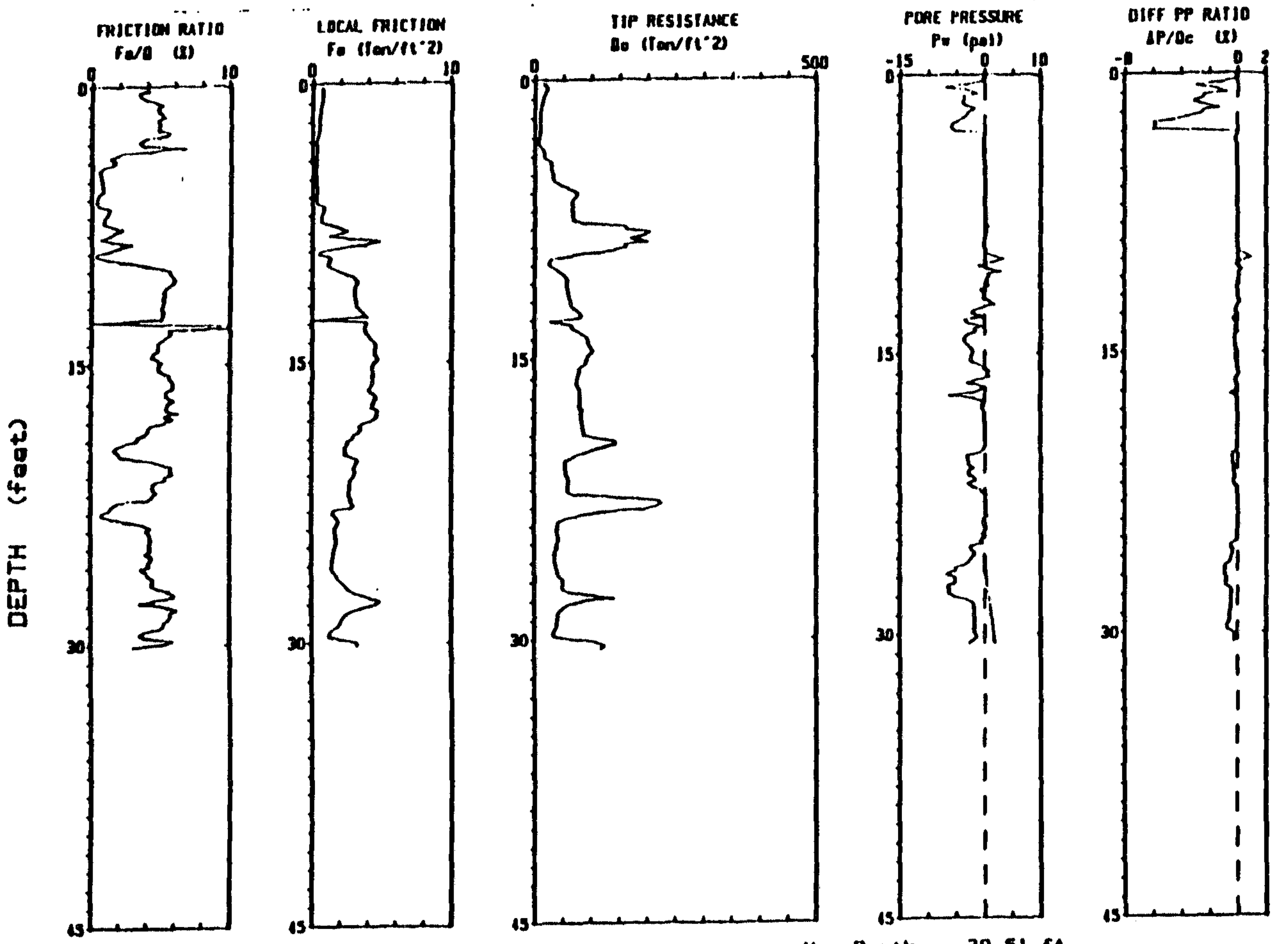
Location : GCPT-33

CPT Date : 03-18-92 11:11

Cone Used : 347TC

Sounding : 920124 Pg 1 / 1

Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.51 ft

BY ____________ DATE ____________

CHECKED BY ____________

FILE 00086-045-003

REVISIONS

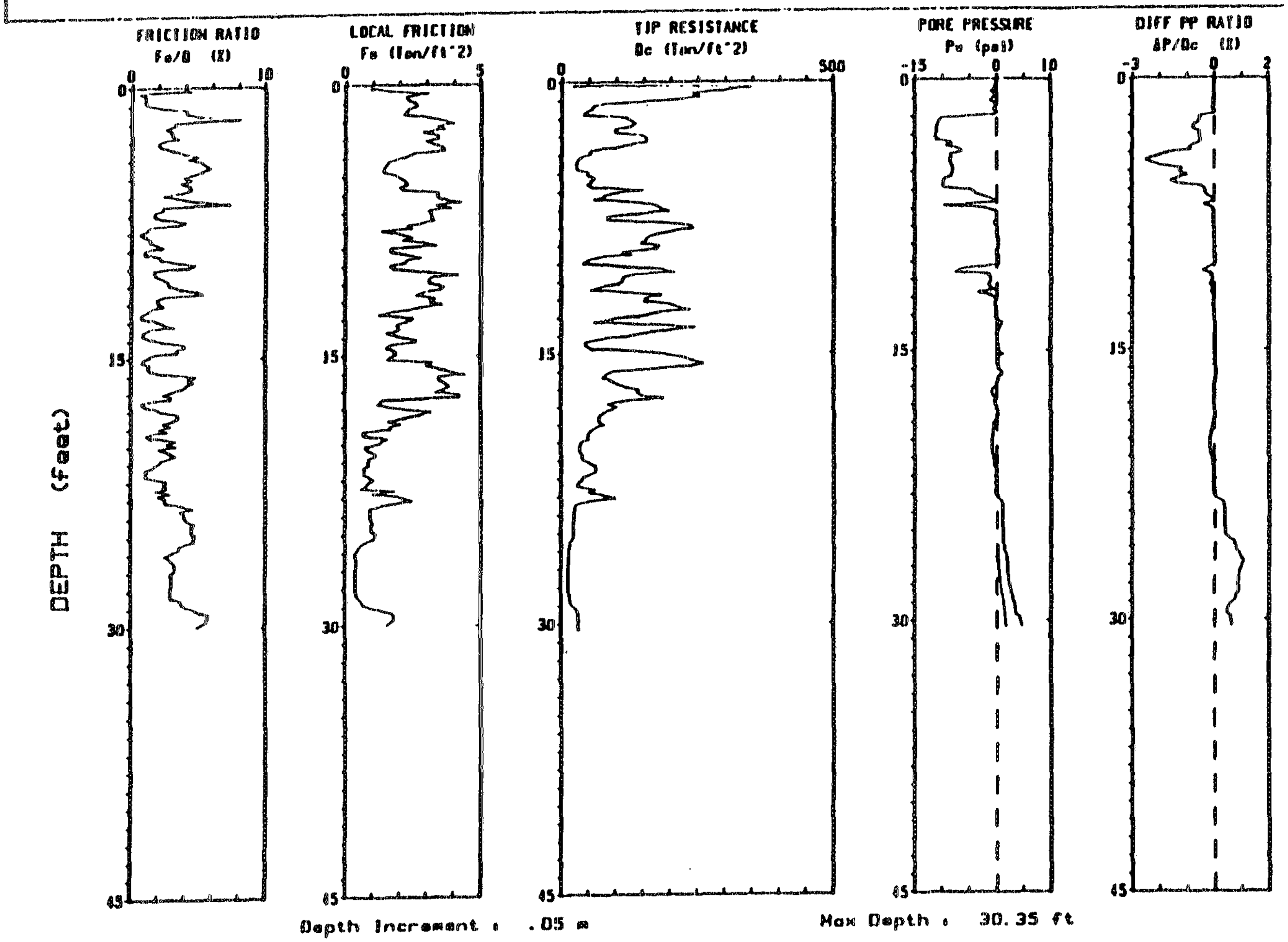
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-34

CPT Date : 03-09-92 13:25
Cone Used : 347TC

Sounding : 920106 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

BY__________ DATE __________
CHECKED BY __________

FILE 00086 045-003

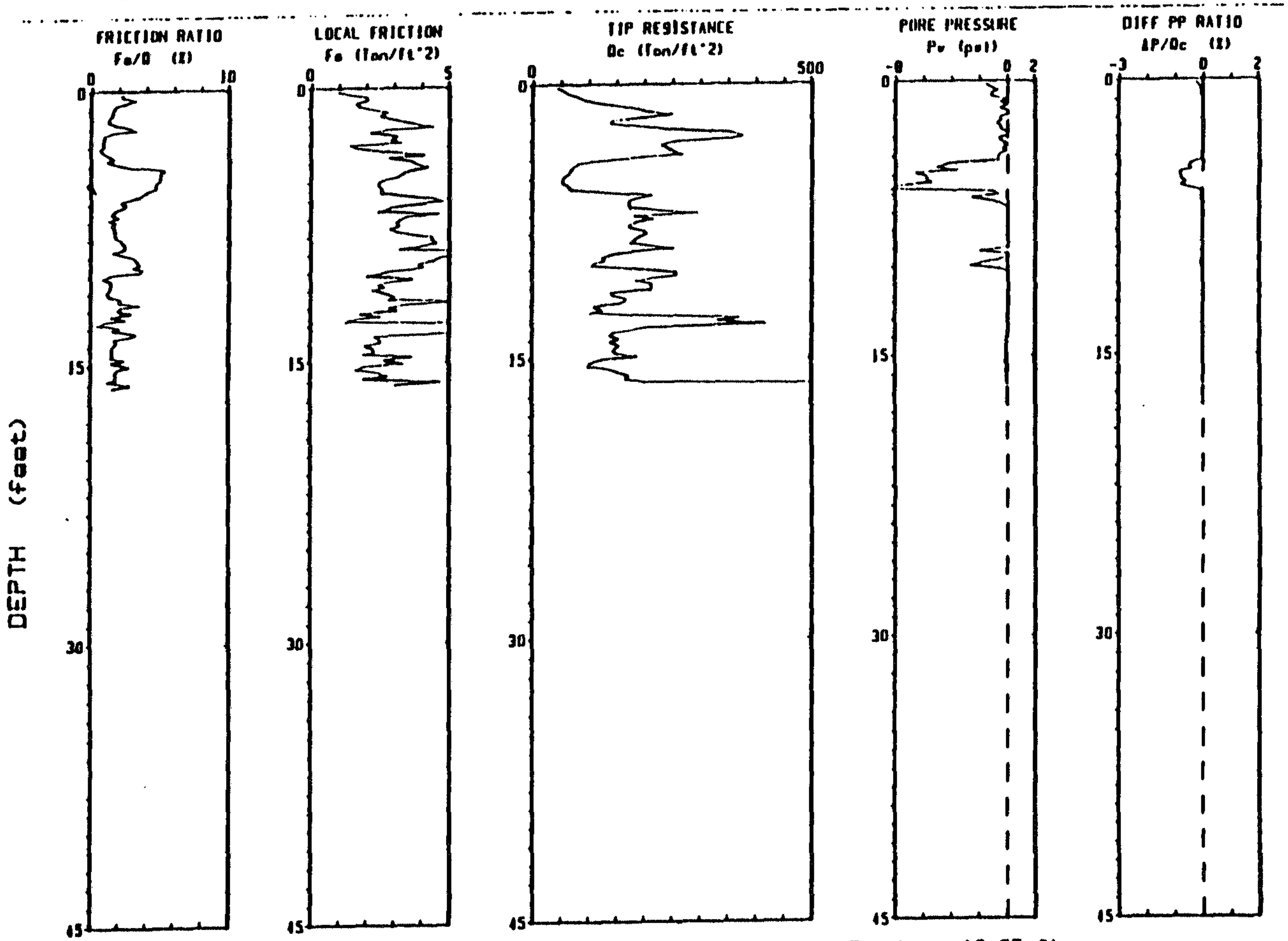
REVISIONS
BY__________ DATE __________

V B I

Operator : CARL G. MORGAN
Location : GCPT-35(A)

CPT Date : 03-02-92 16:41
Cone Used : 347TC

Sounding : 920086 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 16.57 ft

Dames & Moore

BY ____________ DATE ____________

CHECKED BY ____________

FILE 00086-045-003

REVISIONS

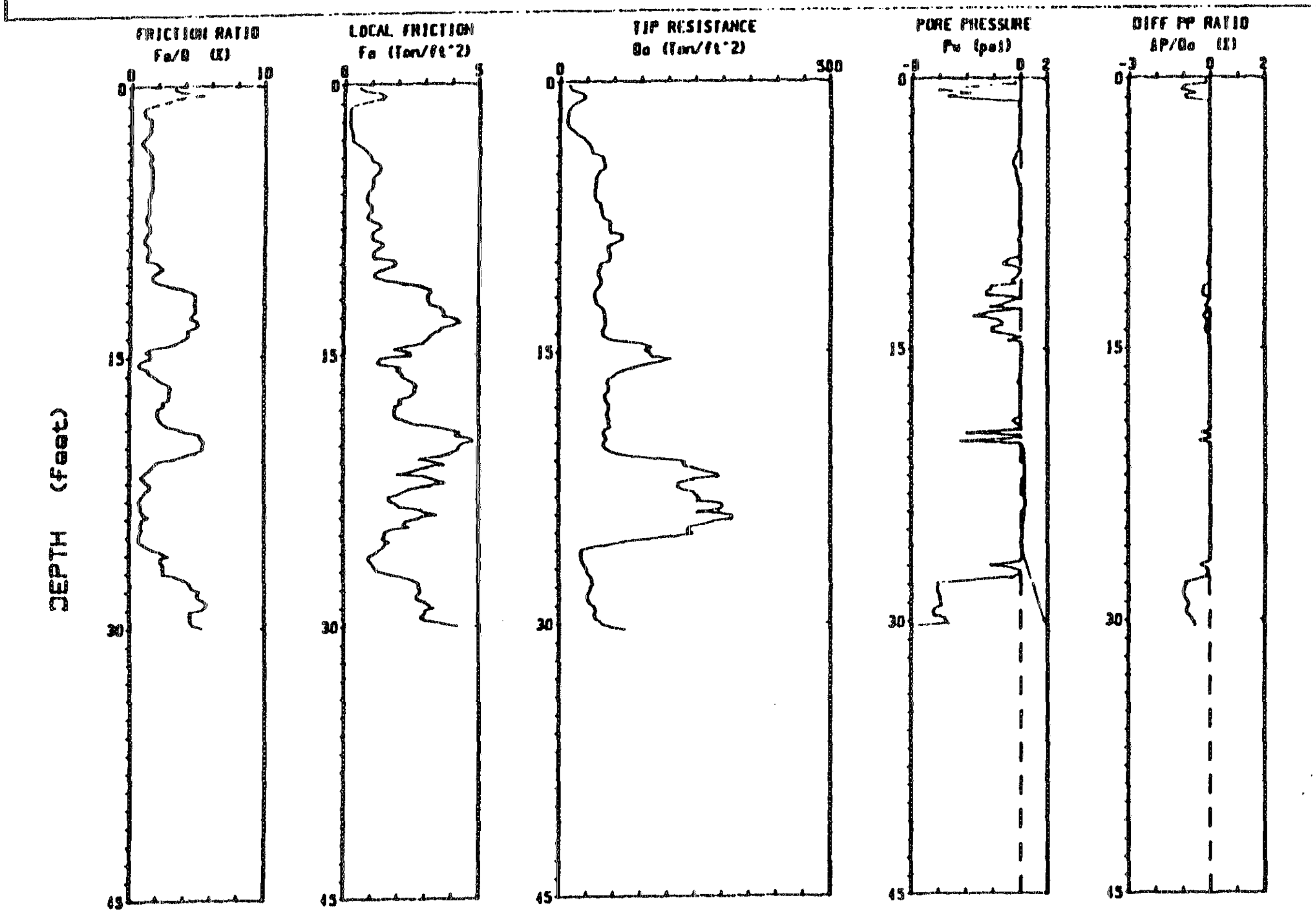
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-358

CPT Date : 03-18-02 09:37
Cone Used : 347TC

Sounding : 920123 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-38

BY ____________ DATE ____________
CHECKED BY ____________

FILE 00086-45-003

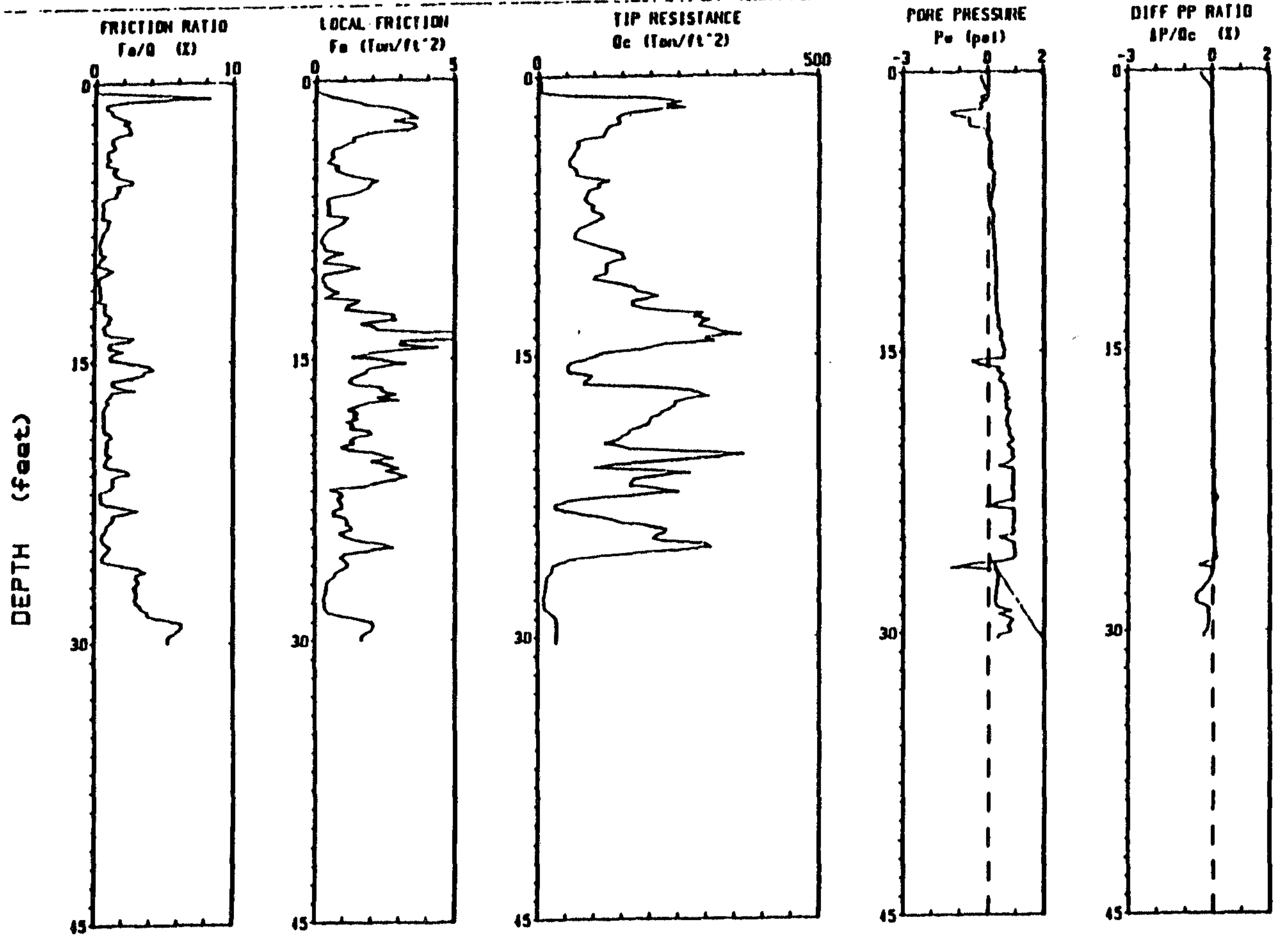
REVISIONS
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-38

CPT Date : 03-08-92 11:42
Cone Used : 347TC

Sounding : 920105 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

J-20

BY __________ DATE __________

CHECKED BY __________

FILE 00086-045-003

REVISIONS

BY ________ DATE ______

V B I

Operator : CARL G. MORGAN

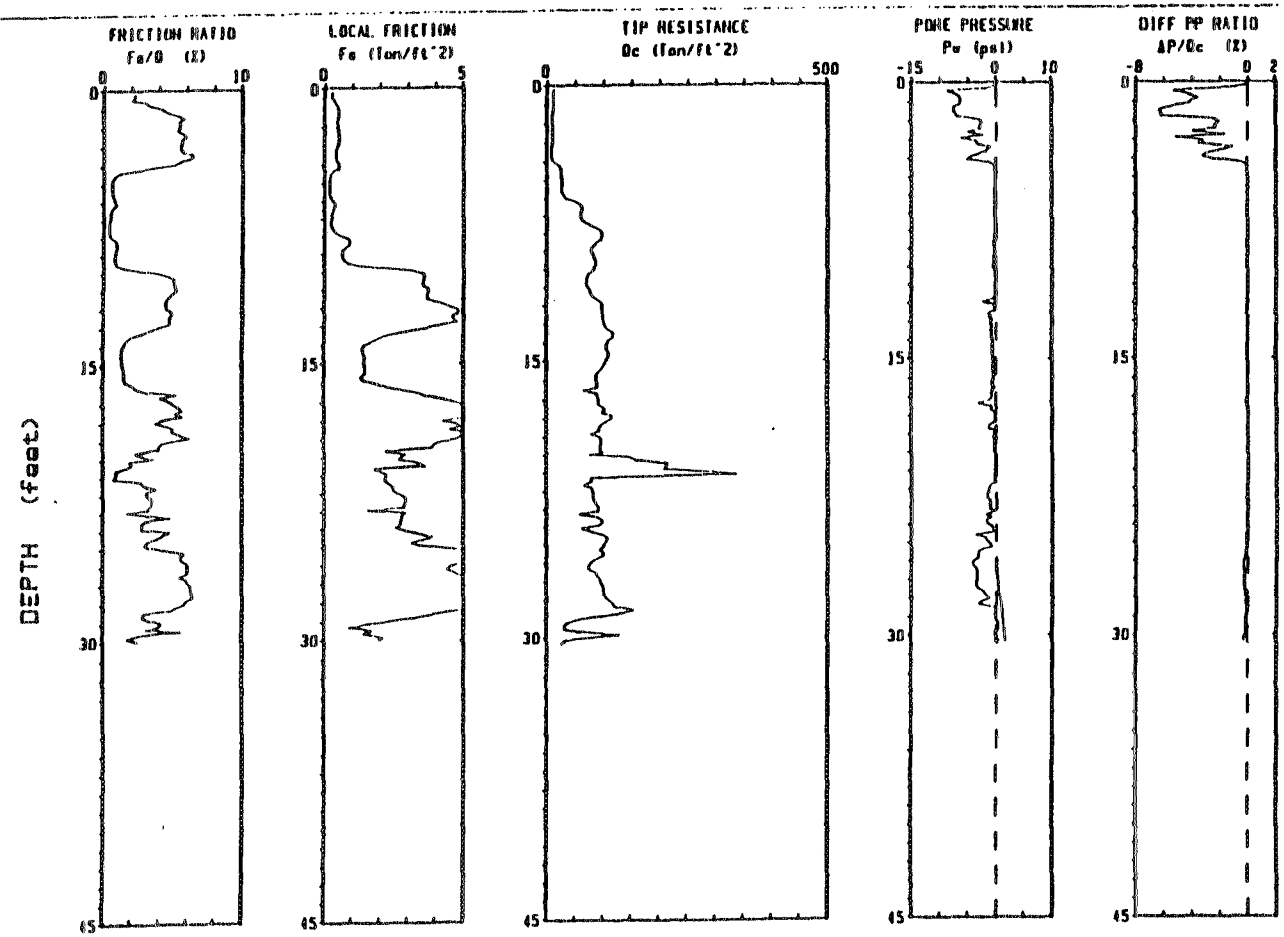
Location : GCPT-37

CPT Date : 03-02-92 14:48

Cone Used : 347TC

Sounding : 920084 Pg 1 / 1

Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-45

BY ____________ DATE ____________
CHECKED BY ____________

FILE 00086 045-003

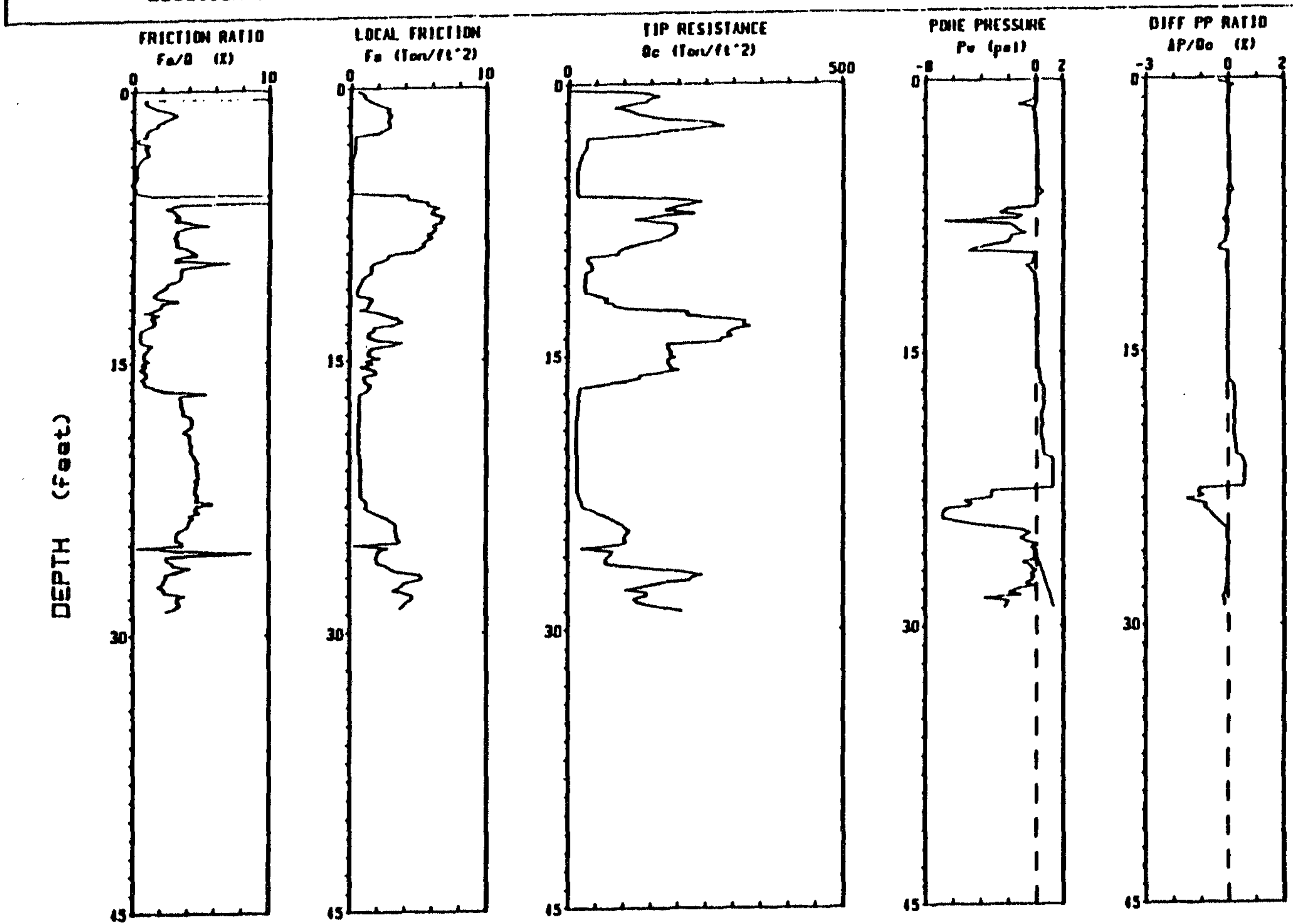
REVISIONS
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-38

CPT Date : 03-08-92 14:07
Cone Used : 347TC

Sounding : 920103 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m
Max Depth : 29.04 ft

Dames & Moore

I-41

BY ____________ DATE ____________

CHECKED BY ____________

FILE 00086-045-003

REVISIONS

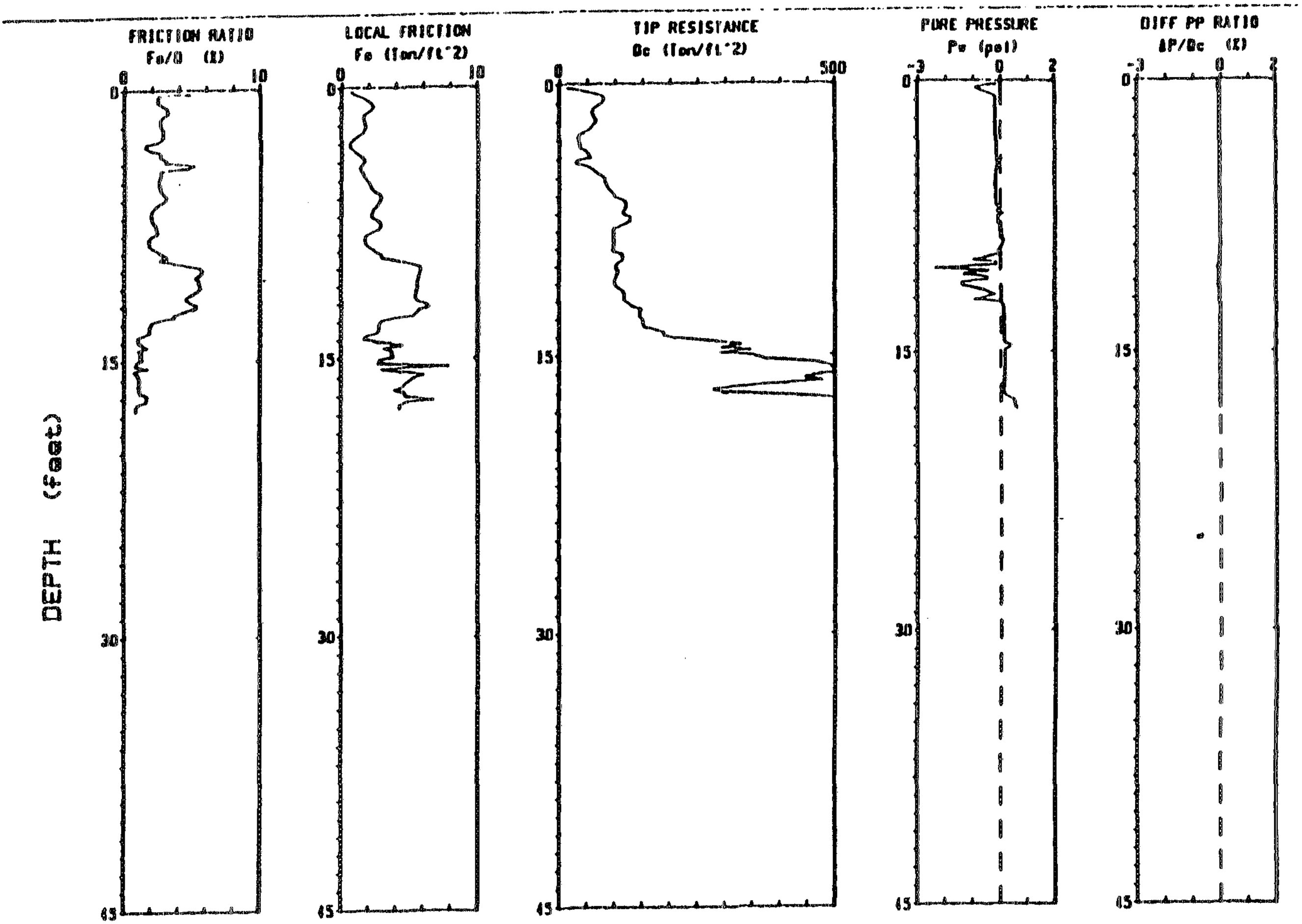
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-38

CPT Date : 03-08-92 15:37
Cone Used : 347TC

Sounding : 920104 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 18.04 ft

BY ____________ DATE ____________
CHECKED BY ____________

FILE 00086 15-003

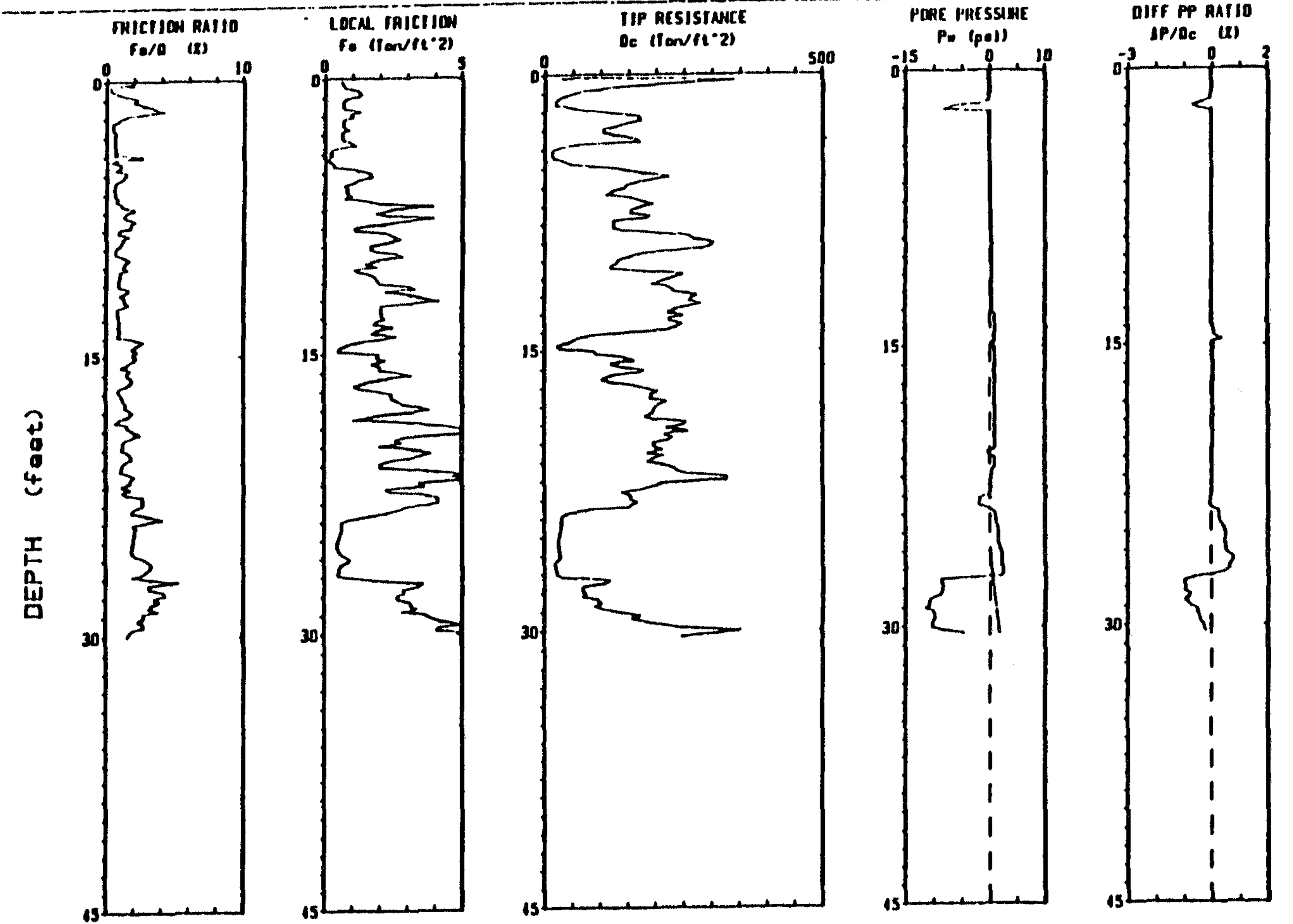
REVISIONS
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-40

CPT Date : 03-06-02 12:25
Cone Used : 347TC

Sounding : 920102 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

I-V-3

BY ______ DATE ______
CHECKED BY ______

FILE 00086-045-003

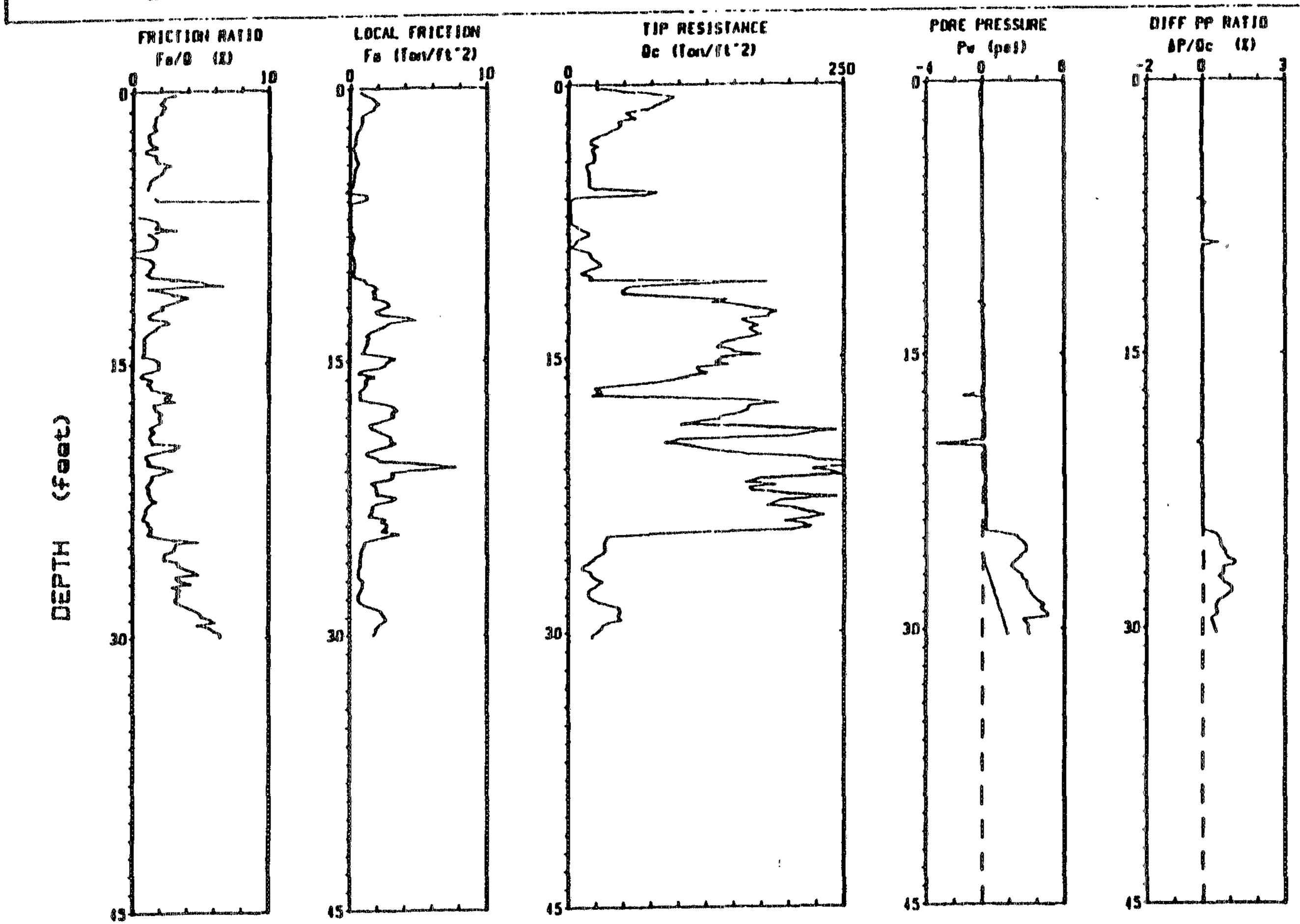
REVISIONS
BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : GCPT-41

CPT Date : 03-06-02 11:16
Cone Used : 347TC

Sounding : 920101 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-44

BY ________ DATE ________

CHECKED BY ________

FILE 00086 045-003

REVISIONS

BY ________ DATE ________

V B I

Operator : CARL G. MORGAN

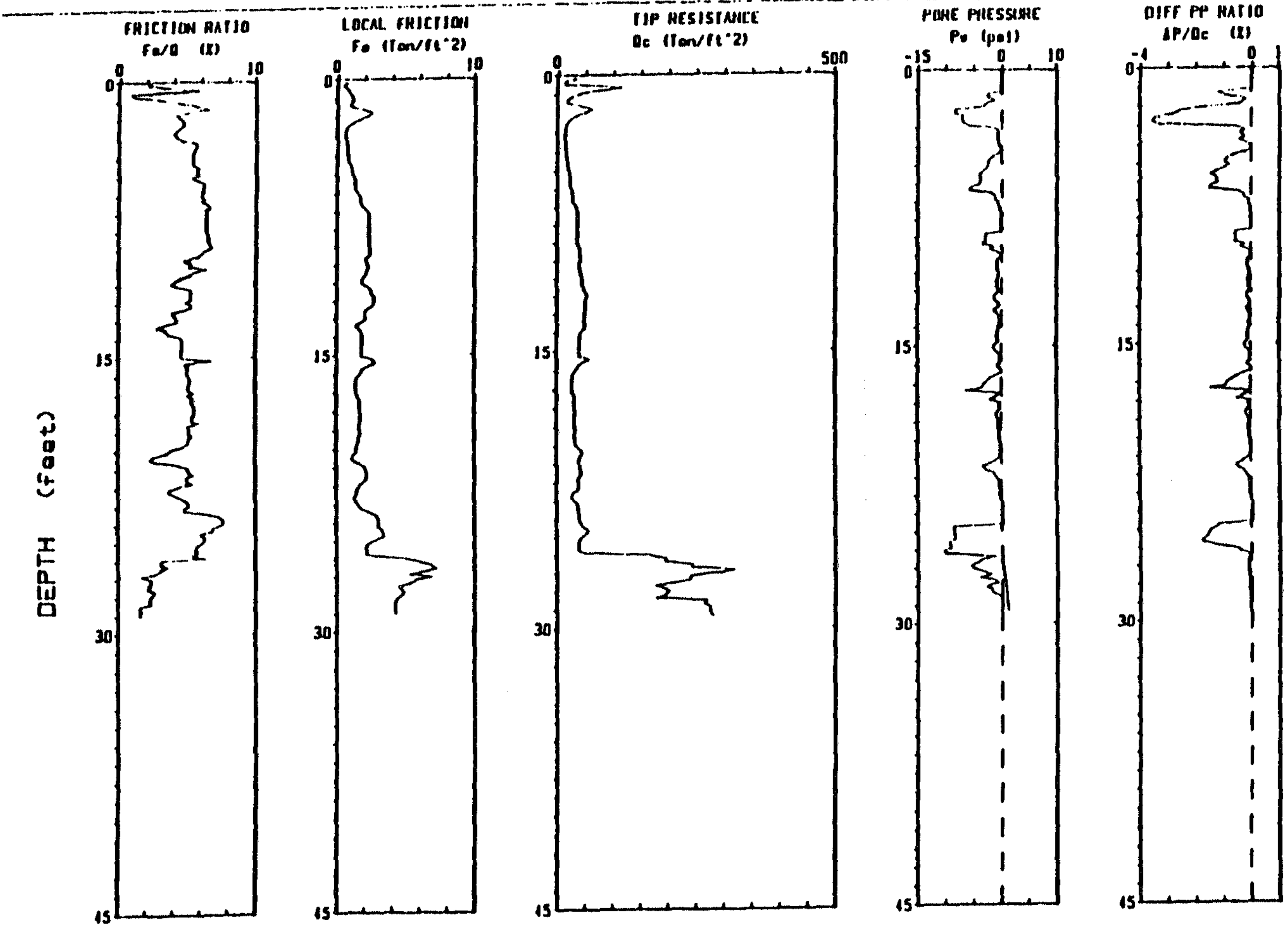
Location : GCPT-42

CPT Date : 03-06-82 00:45

Cone Used : 347TC

Sounding : 820100 Pg 1 / 1

Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 29.36 ft

Dames & Moore

A-45

BY ____________ DATE ____________

CHECKED BY ____________

FILE 00086-045-003

REVISIONS

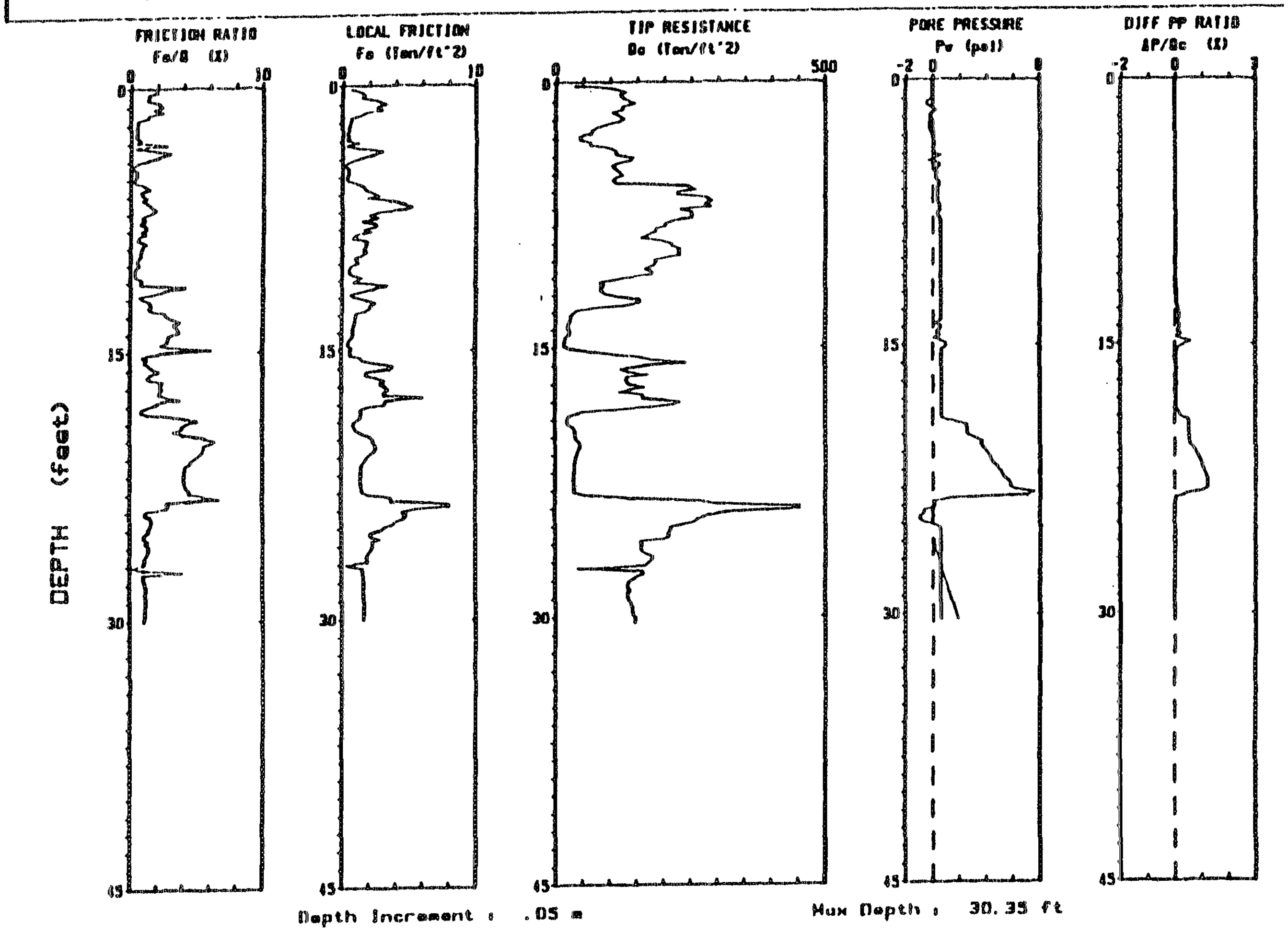
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : DP-4

CPT Date : 03-18-02 12:40
Cone Used : 347TC

Sounding : 020120 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

BY ______ DATE ______
CHECKED BY ______

FILE 00086-15-003

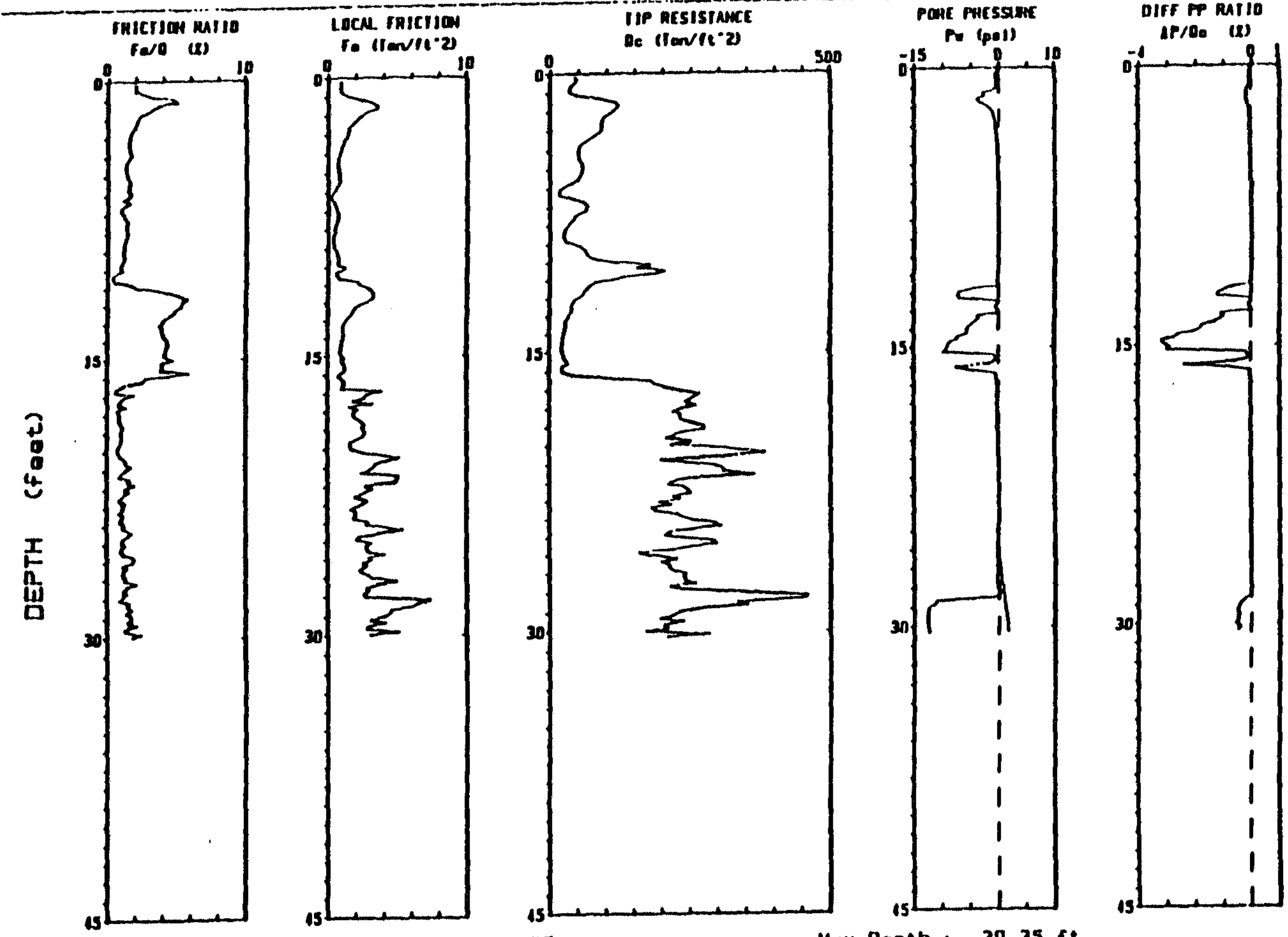
REVISIONS
BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : OP-5

CPT Date : 03-16-82 13:42
Cone Used : 347TC

Sounding : 820121 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

BY__________ DATE __________
CHECKED BY __________

FILE 00086 - 045 - 003

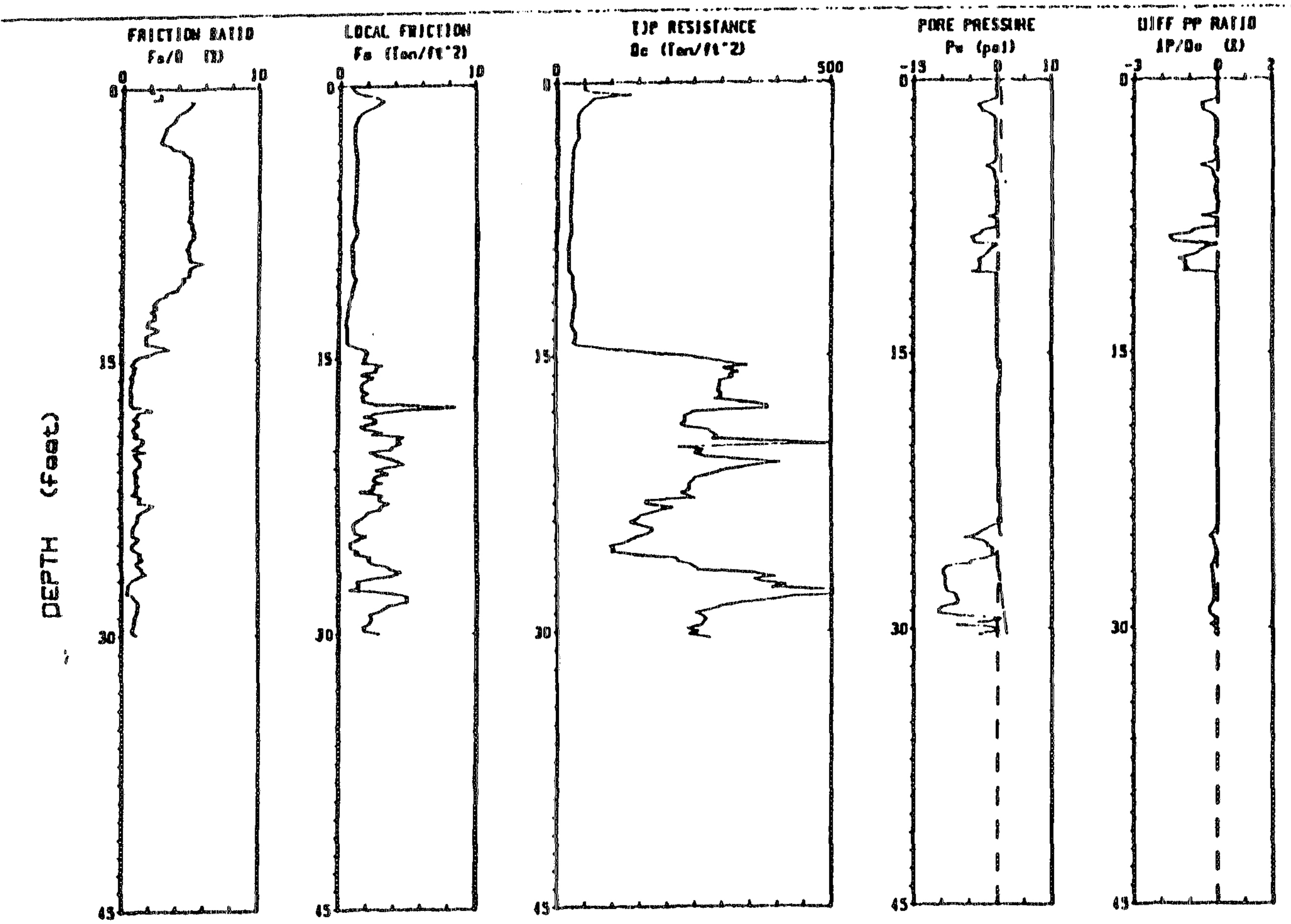
REVISIONS
BY__________ DATE __________

V B I

Operator : CARL G. MORGAN
Location : DP-8

CPT Date : 03-18-92 14:30
Cone Used : 3471C

Sounding : 920122 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-48

BY ____________ DATE ____________
CHECKED BY ____________

FILE 000 045 - 003

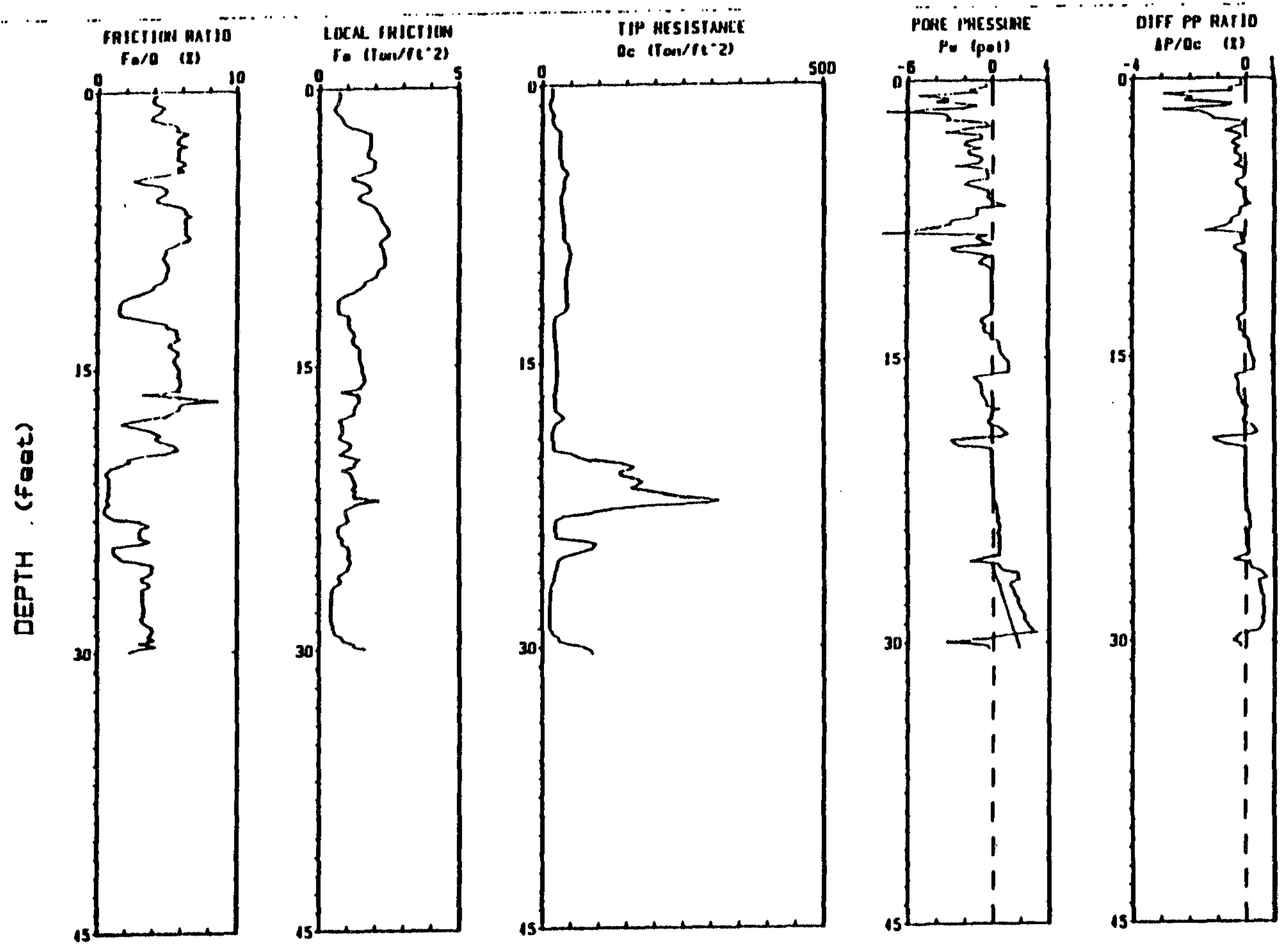
REVISIONS
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : CCPT-1

CPT Date : 03-03-02 12:02
Cone Used : 347TC

Sounding : 020089 Pg 1 / 1
Job No. : 0088-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

BY ____________ DATE ____________

CHECKED BY ____________

FILE 00086-045-003

REVISIONS

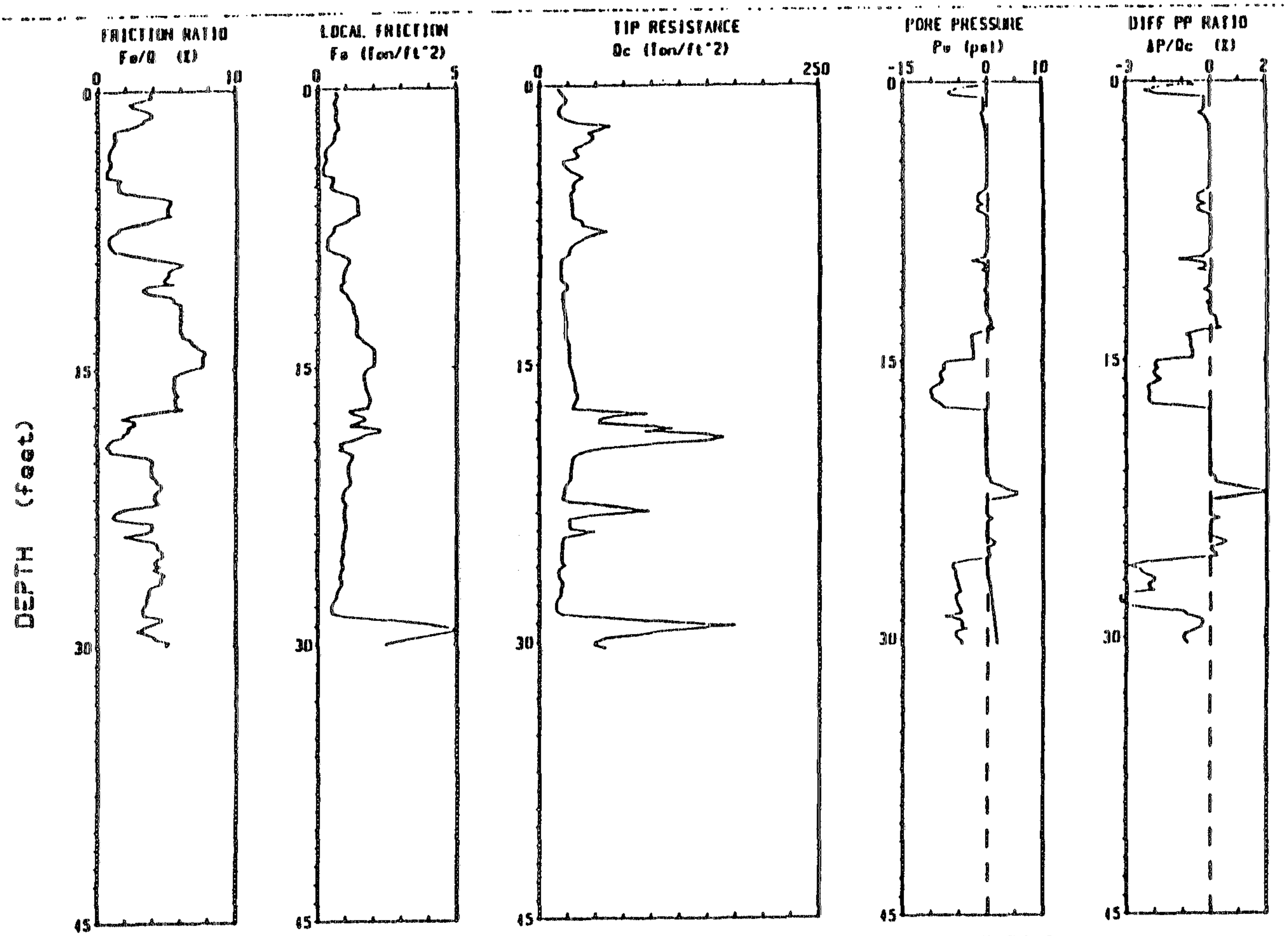
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : CCPT-2

CPT Date : 03-03-02 00:40
Cone Used : 3471C

Sounding : 020007 Pg 1 / 1
Job No. : 0080-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-50

BY ______ DATE ______
CHECKED BY ______

FILE 000Bc 045-003

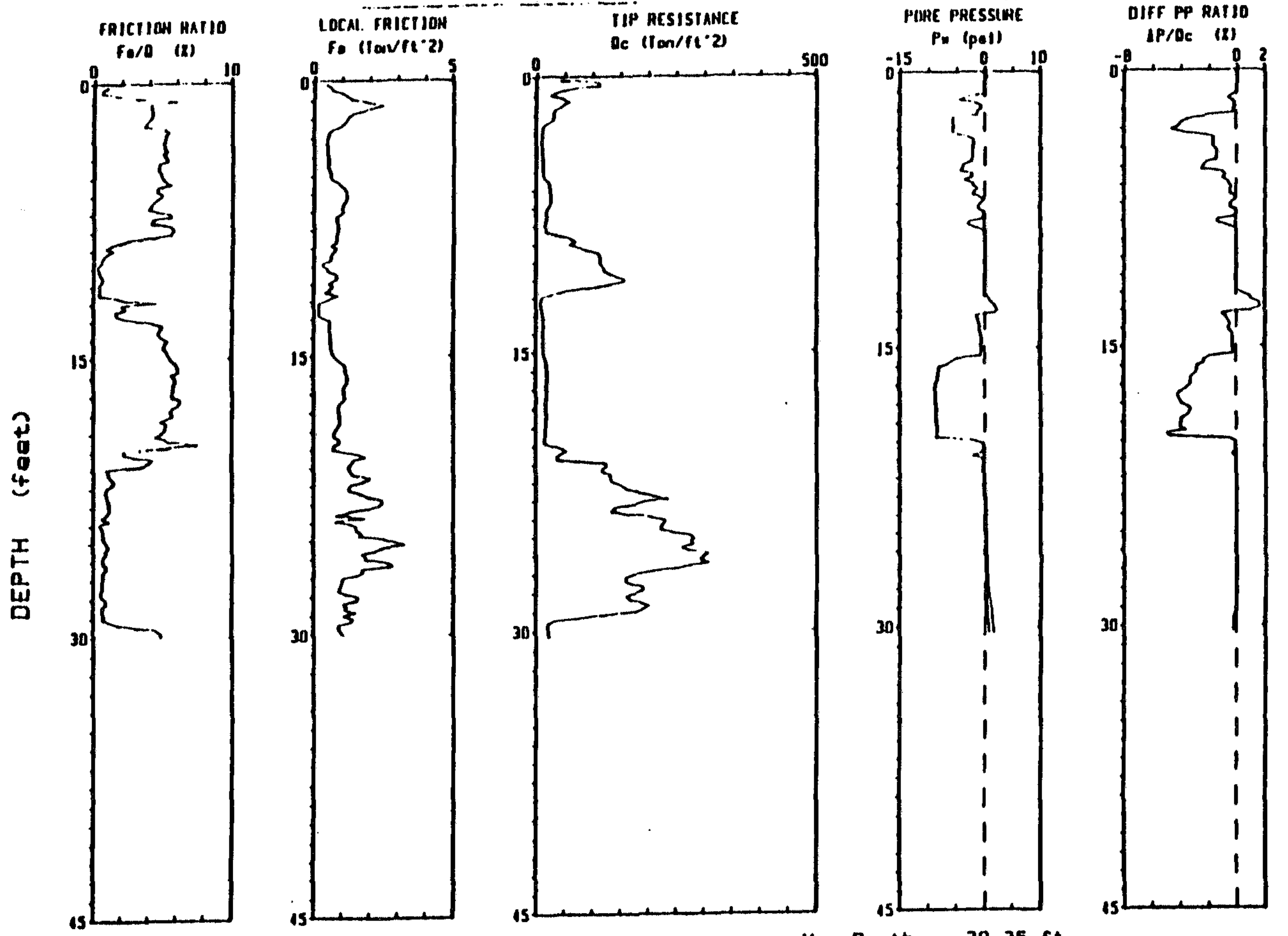
REVISIONS
BY ______ DATE ______

V B I

Operator : CARL G. MORGAN
Location : CCPT-5

CPT Date : 03-03-02 15:02
Cone Used : 347TC

Sounding : 920091 Pg 1 / 1
Job No. : 0088-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

BY__________ DATE __________

CHECKED BY __________

FILE 00086-045-003

REVISIONS

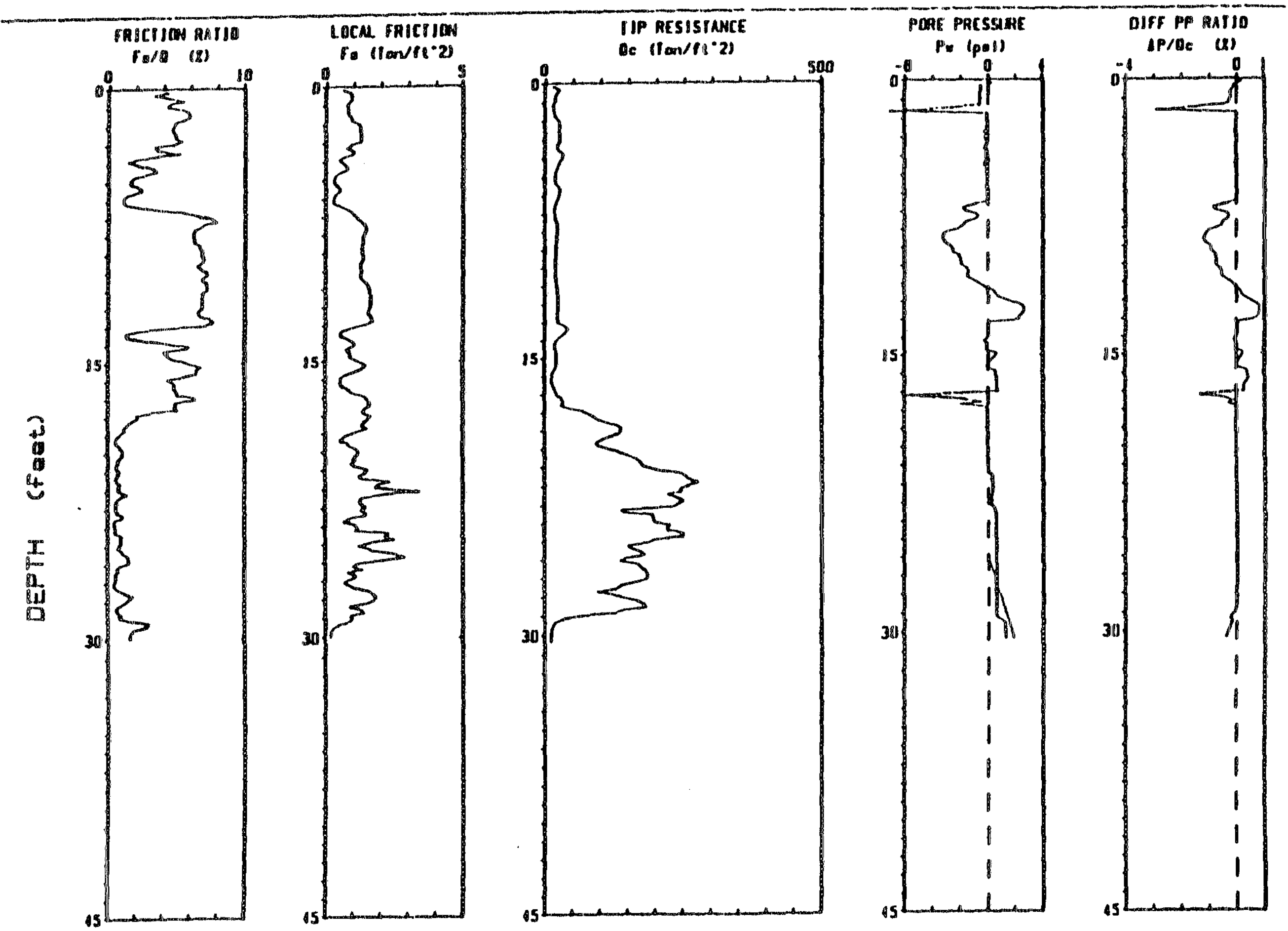
BY__________ DATE __________

V B I

Operator : CARL G. MORGAN
Location : OP-7

CPT Date : 03-03-02 10:49
Cone Used : 347TC

Sounding : 020008 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-52

BY ______ DATE ______
CHECKED BY ______

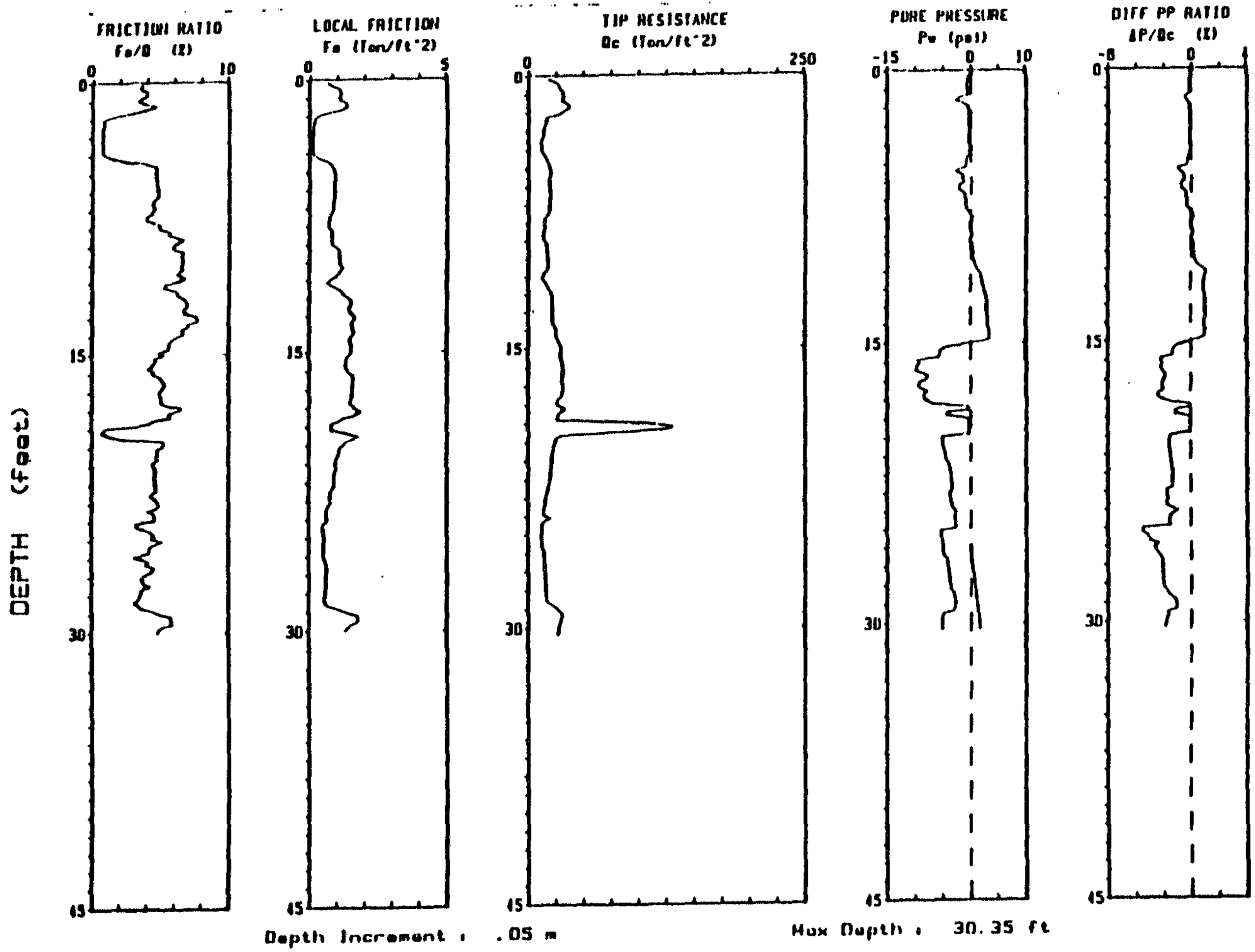
FILE 0008. 045 - 003

REVISIONS
BY ______ DATE ______

Operator : CARL G. MORGAN
Location : OP-8

CPT Date : 03-03-02 13:31
Cone Used : 347TC

Sounding : 020000 Pg 1 / 1
Job No. : 0008-045-003



Depth Increment : .05 m

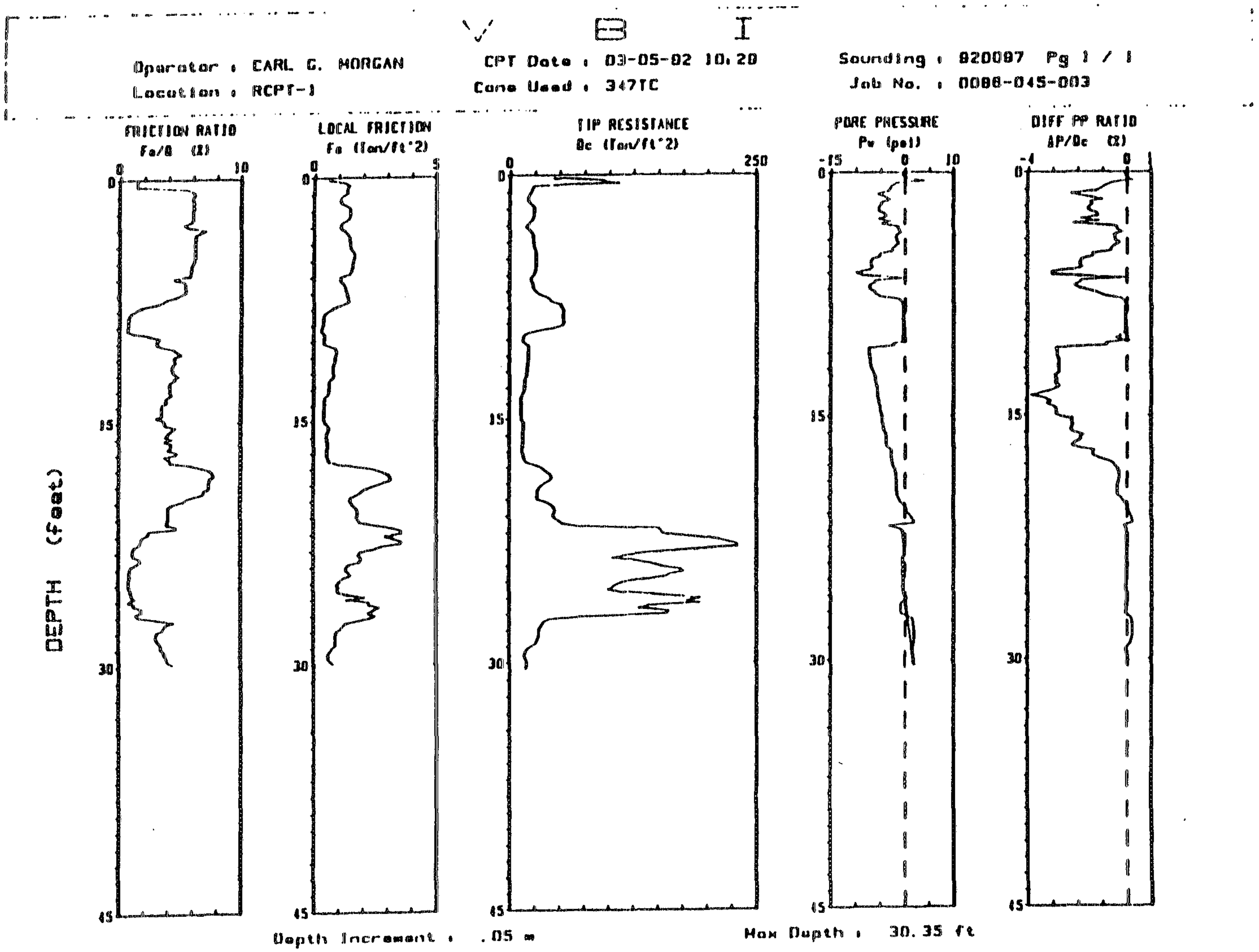
Max Depth : 30.35 ft

Dames & Moore

BY ____________ DATE ____________
CHECKED BY ____________

FILE 00086-045-003

REVISIONS
BY ________ DATE ______



V B I

Operator : CARL G. MORGAN
Location : RCPT-1

CPT Date : 03-05-82 10:20
Cone Used : 347TC

Sounding : 820097 Pg 1 / 1
Job No. : 0088-045-003

Depth Increment : .05 m
Max Depth : 30.35 ft

Dames & Moore

BY__________ DATE __________
CHECKED BY __________

FILE 000[illegible]-045-003

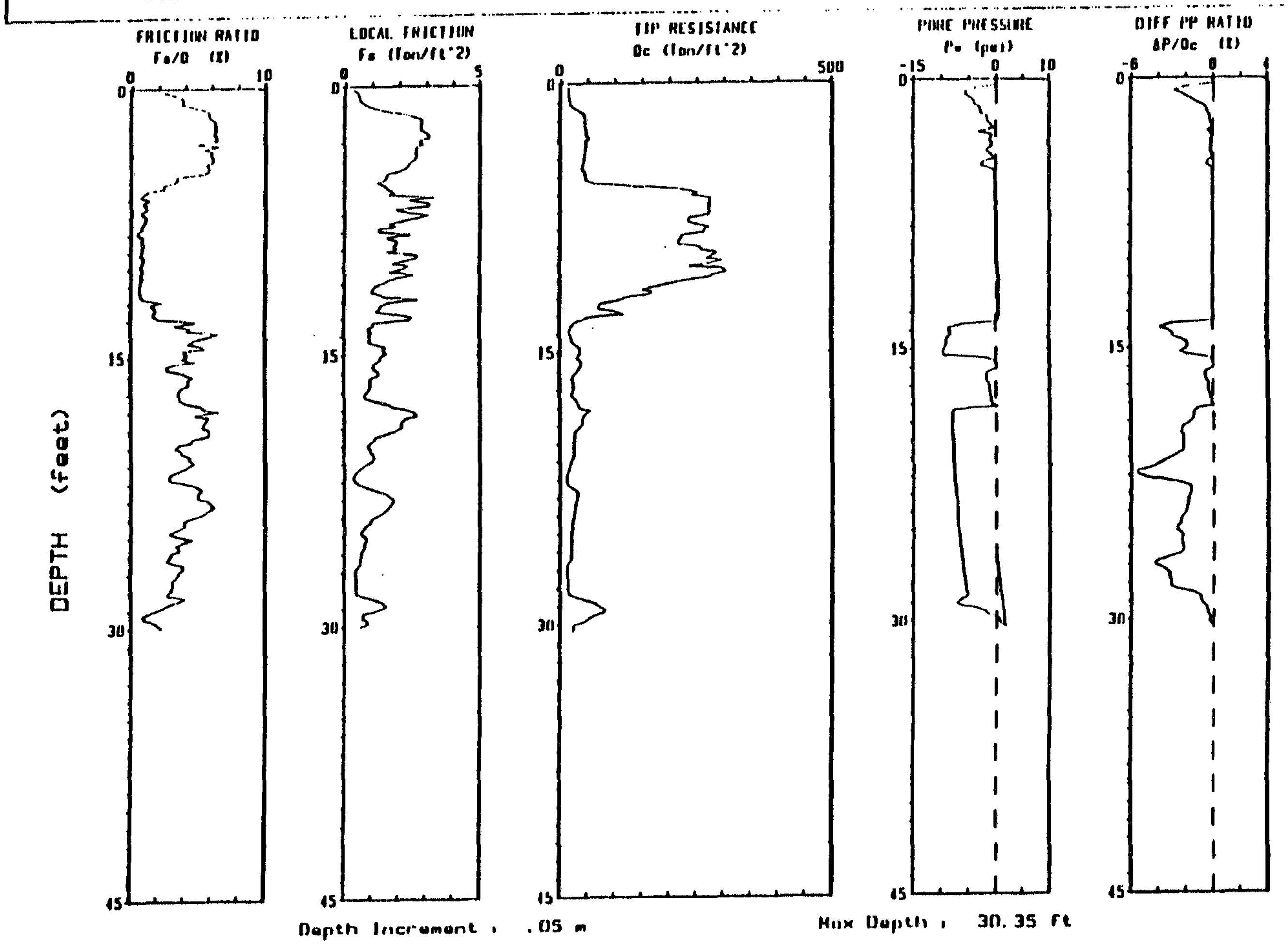
REVISIONS
BY__________ DATE __________

V B I

Operator : CARL G. MORGAN
Location : RCPT-3

CPT Date : 03-04-92 12:09
Cone Used : 3471C

Sounding : 920094 Pg 1 / 1
Job No. : 0086-045-003



Dames & Moore

A-55

BY __________ DATE __________
CHECKED BY __________

FILE 00086-045-003

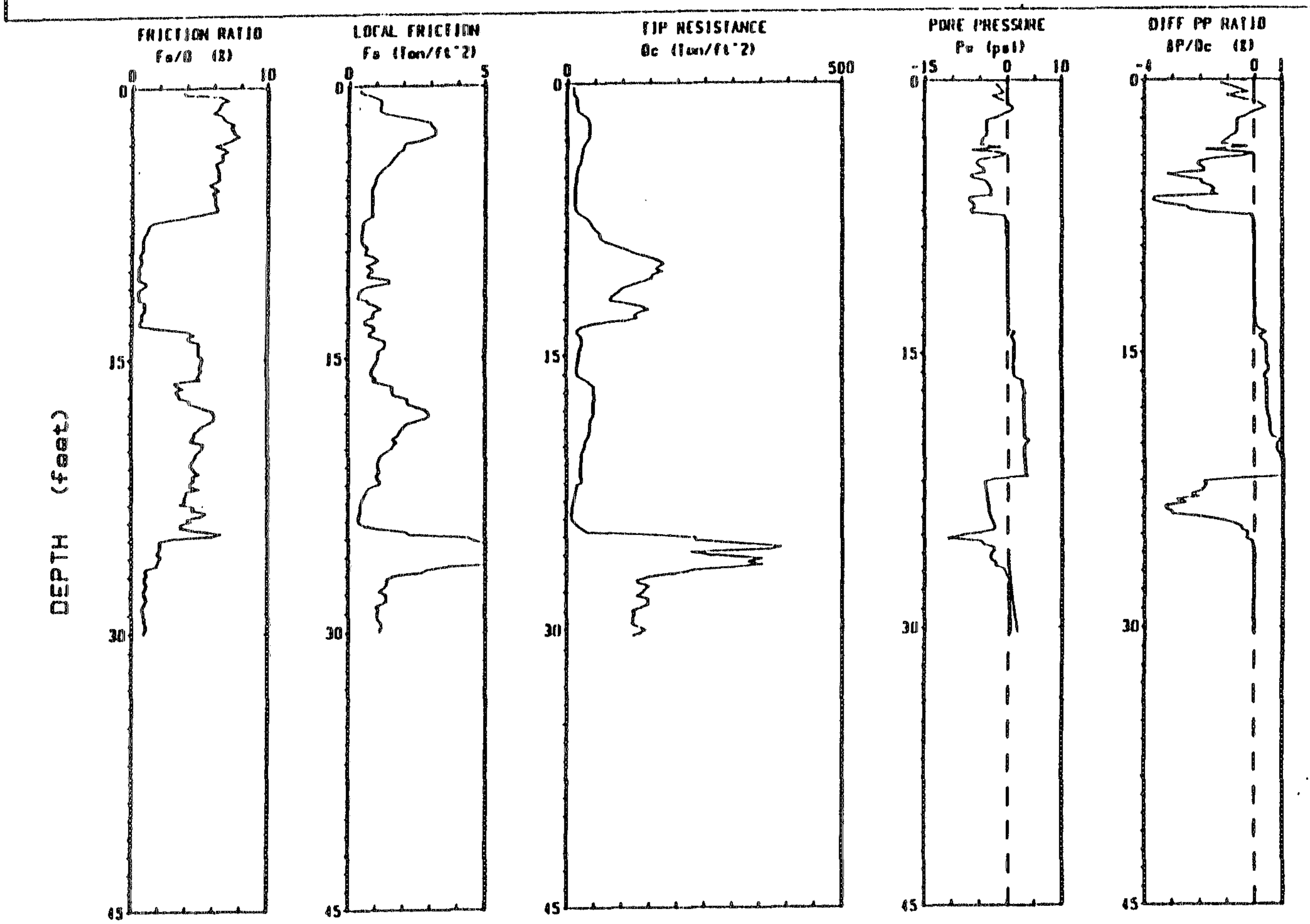
REVISIONS
BY __________ DATE __________

V B I

Operator : CARL G. MORGAN
Location : RCPT-3A

CPT Date : 03-04-82 13:33
Cone Used : 347TC

Sounding : 020095 Pg 1 / 1
Job No. : 0006-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

BY __________ DATE __________
CHECKED BY __________

FILE 00086-045-003

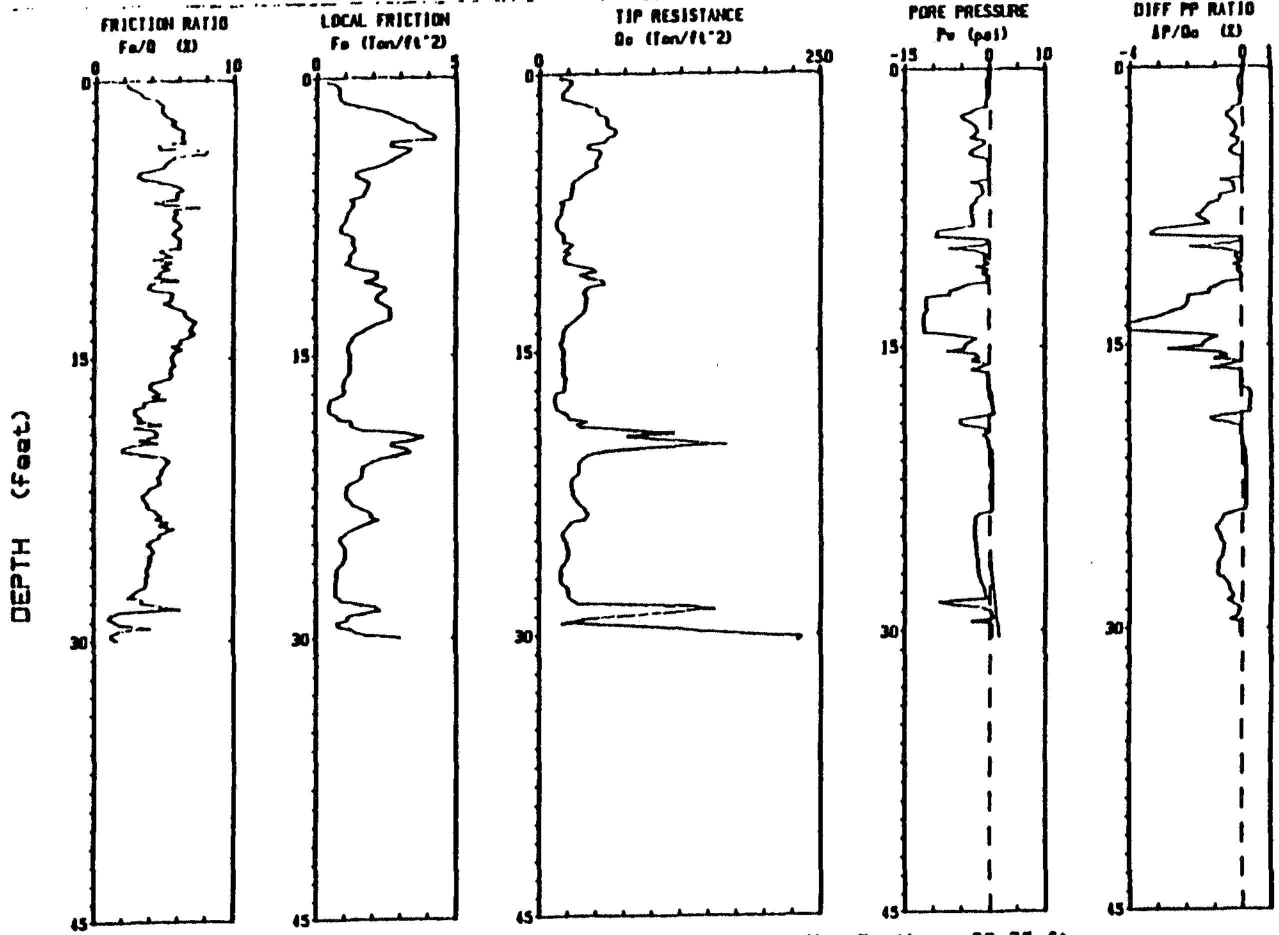
REVISIONS
BY __________ DATE __________

V B I

Operator : CARL G. MORGAN
Location : RCPT-4

CPT Date : 03-18-92 12:38
Cone Used : 347TC

Sounding : 920125 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m
Max Depth : 30.35 ft

BY ____________ DATE ____________

CHECKED BY ____________

FILE 00086-045-003

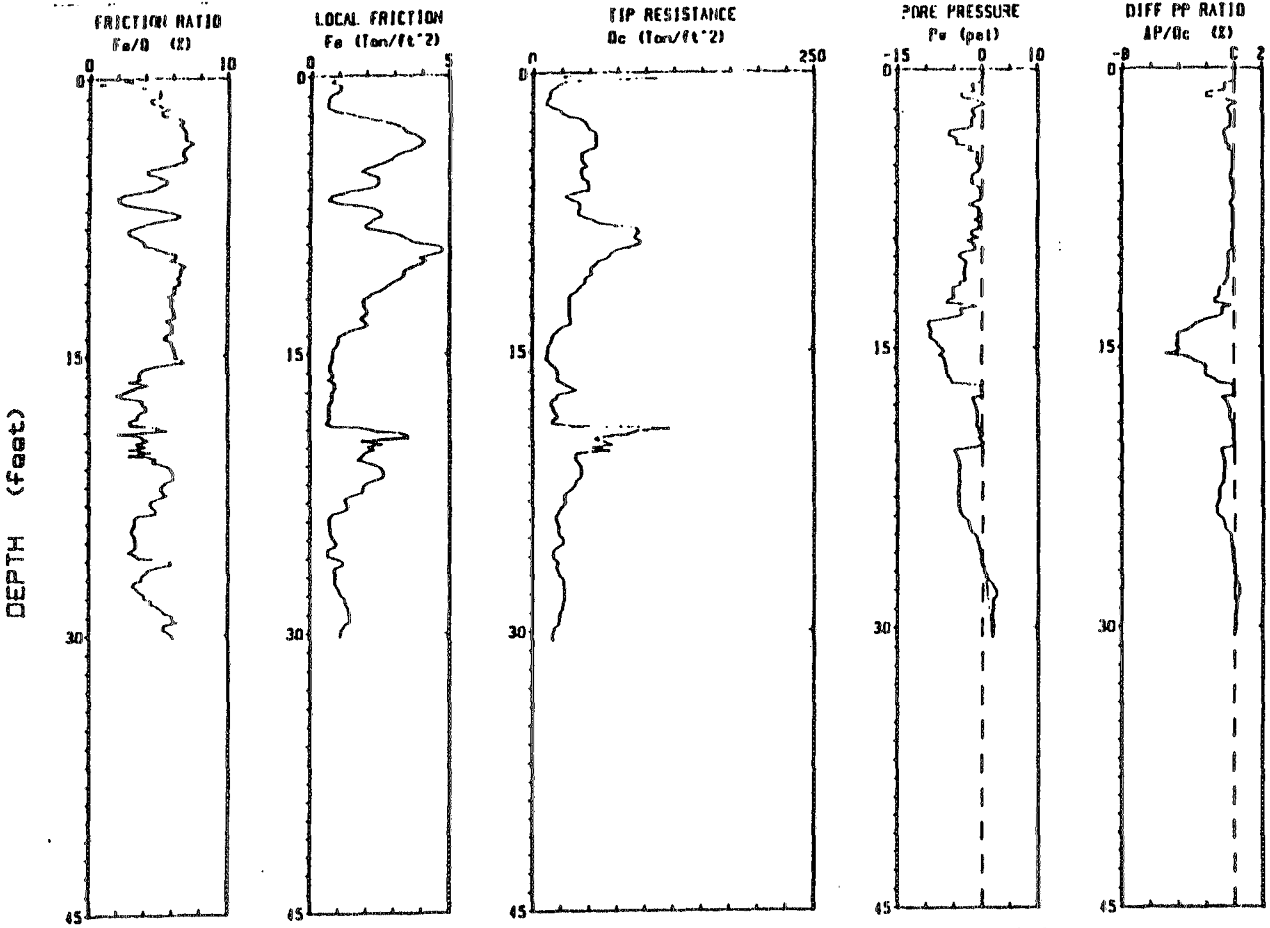
REVISIONS

BY ________ DATE ______

Operator : CARL G. MORGAN
Location : RCPT-5

CPT Date : 03-04-92 09:38
Cone Used : 347TC

Sounding : 920092 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.51 ft

Dames & Moore

BY ____________ DATE ____________

CHECKED BY ________________

FILE 00086-045-003

REVISIONS

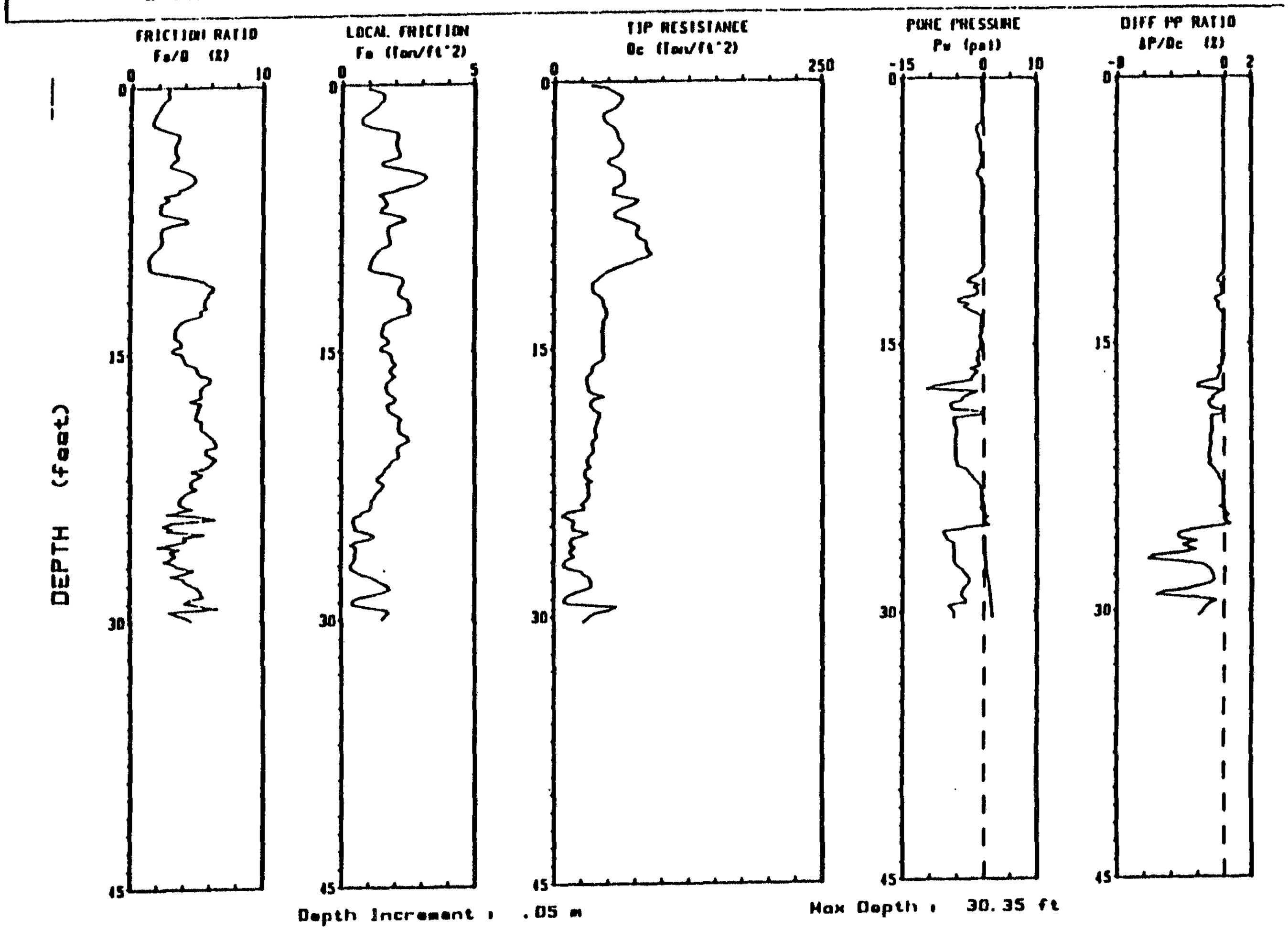
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : OP-8

CPT Date : 03-05-02 12:05
Cone Used : 347TC

Sounding : 020098 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

BY ____________ DATE ____________
CHECKED BY ____________

FILE 00086-045-003

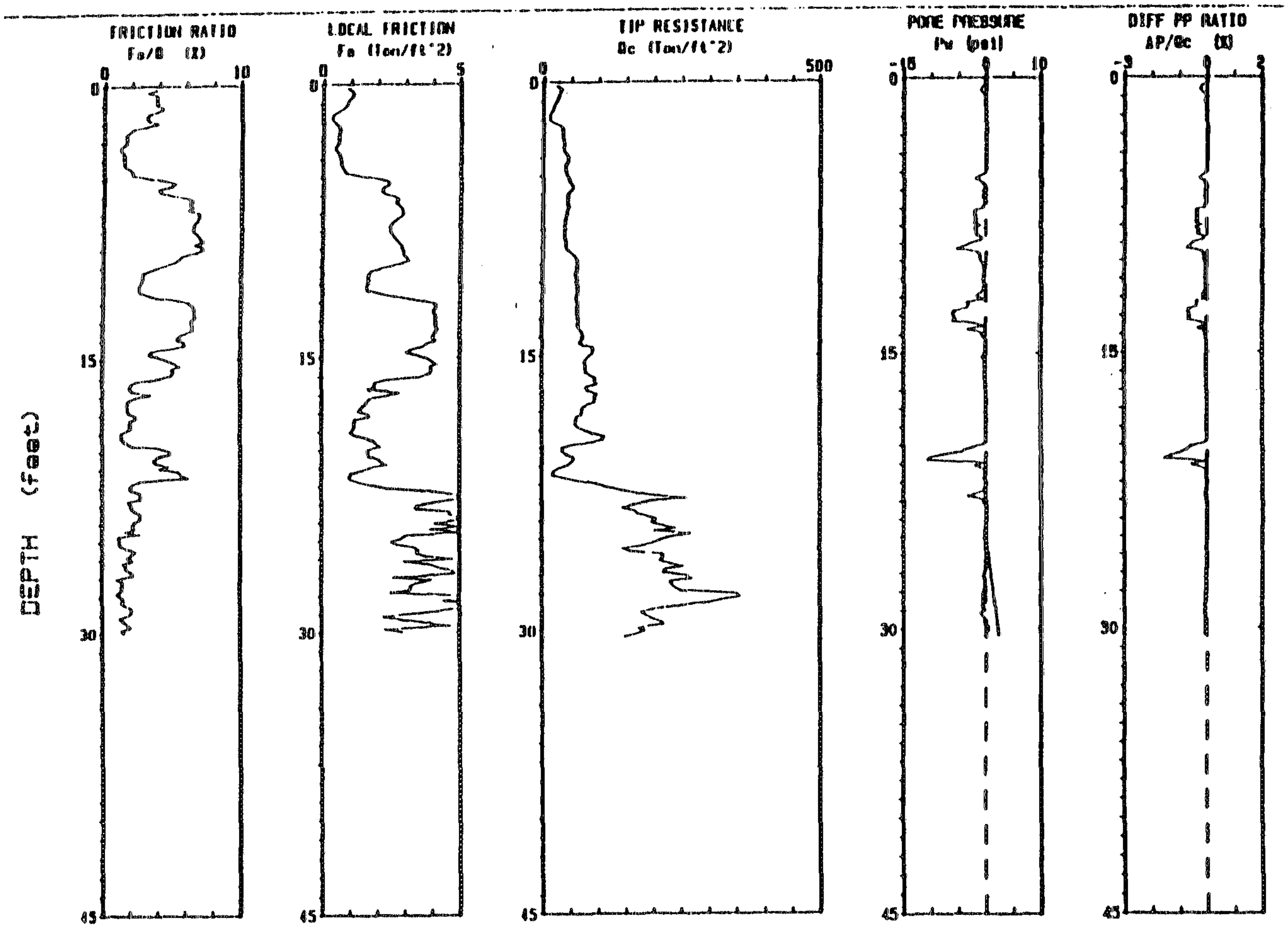
REVISIONS
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : OP-10

CPT Date : 03-04-92 14:40
Cone Used : 347TC

Sounding : 020098 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m
Max Depth : 30.35 ft

BY__________ DATE __________
CHECKED BY __________

FILE 0008c 045 - 003

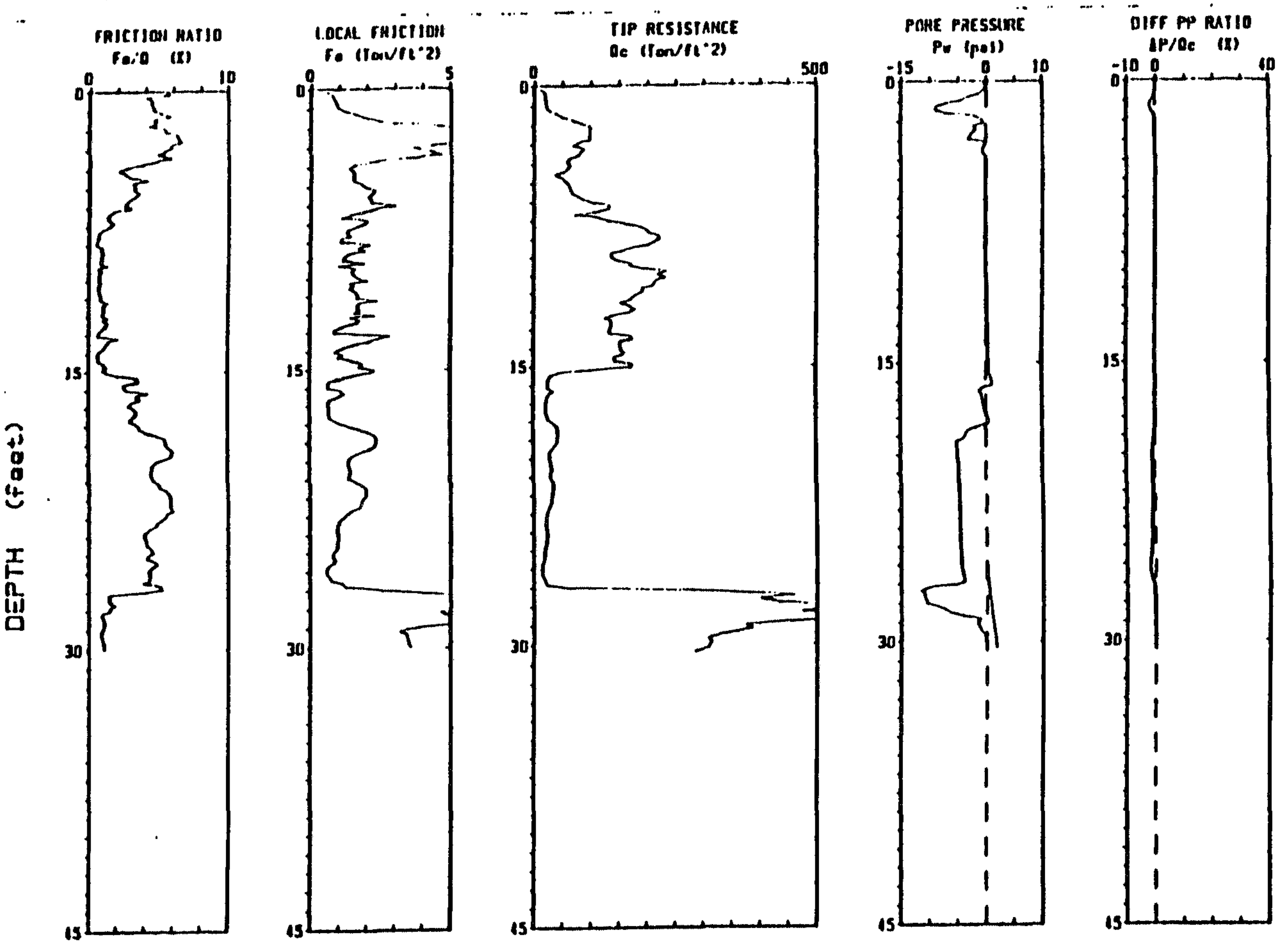
REVISIONS
BY________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : OP-11

CPT Date : 03-04-92 10:51
Cone Used : 347TC

Sounding : 920093 Pg 1 / 1
Job No. : 0088-045-003



Depth Increment : .05 m
Max Depth : 30.35 ft

BY ____________ DATE ____________

CHECKED BY ____________

FILE 00086-045-003

REVISIONS

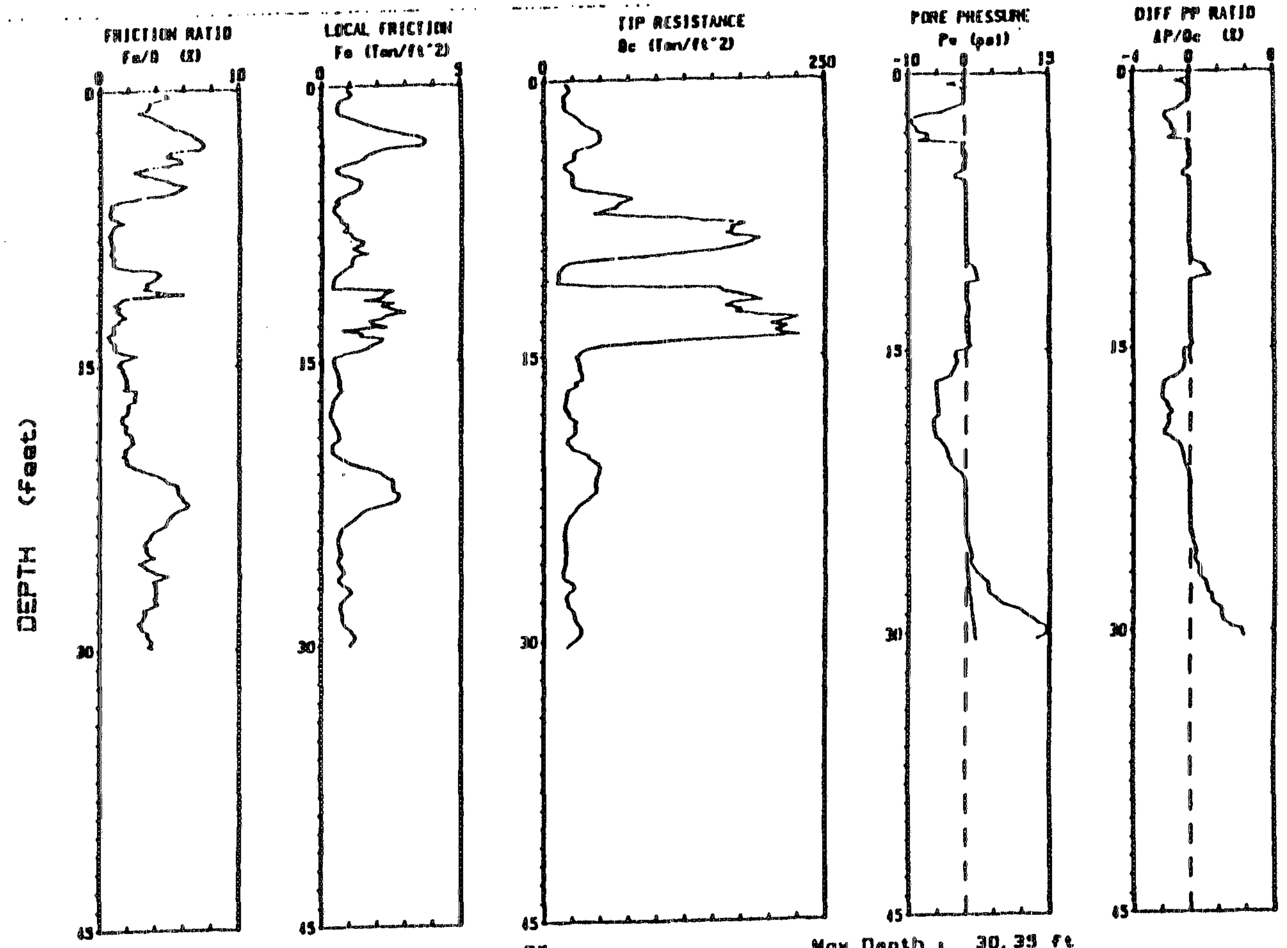
BY ________ DATE ______

V B I

Operator : CARL G. MORGAN
Location : DP-12

CPT Date : 03-18-92 11:11
Cone Used : 347TC

Sounding : 920118 Pg 1 / 1
Job No. : 0086-045-003



Depth Increment : .05 m

Max Depth : 30.35 ft

Dames & Moore

A-62

ATTACHMENT B

LOG OF BORINGS

UPPER GUADALUPE RIVER PROJECT

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. GR-93-2

BORING LOCATION: In the street in front of 1593 Mackey Avenue

DATE STARTED: 5/28/93 **DATE FINISHED:** 5/28/93

DRILLING METHOD: 4 7/8-inch solid auger, drag bit and rock bit

HAMMER WEIGHT: 140 lbs **DROP:** 30 inches

SAMPLER: Shelby tube 3" x 36"/SPT

NOTES:
Drilling equipment: Failing 1500
Drilling contractor: Pitcher Drilling - Ron Baker
Logged by: E. Chase

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 116.0	REMARKS
1				LEAN CLAY (CL) Soft, brown, moist	Start drilling 7:30
2					
3					
4	1	SPT	2		
5					
6					
7					
8	2	Shelby	200 psi	Medium stiff	
9				Dark olive brown	
10			250 psi		
11	3	Shelby			
12			300 psi	Stiff	
13					
14					
15					
16				Olive gray	
17					
18					
19					
20					
21	4	Shelby	200 psi		
22					
23				Bottom of boring at 22.5 feet	End drilling 8:30. Begin pull-out
24					
25					
26					
27					
28					
29					
30					

ʒ-1-88

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. GR-93-3

BORING LOCATION: San Jose Water District Yard
DATE STARTED: 5/27/93
DATE FINISHED: 5/27/93
DRILLING METHOD: 4 7/8" solid auger/rotary wash - drag bit and rock bit
HAMMER WEIGHT: 140 lbs
DROP: 30 inches
SAMPLER: Shelby tube 3" x 36"/SPT

NOTES:
Drilling equipment: Failing 1500
Drilling contractor: Pitcher Drilling - Ron Baker
Drilling fluid: 2 sacks Super Gel-X and water
Logged by: E. Chase

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 126.5	REMARKS
1				SILTY SAND (SM) Loose, brown, slightly moist, occasional gravels, fine sand	Start drilling 1:15
2					
3					Casing placed to 6 feet
4	1	SPT	3		
5	2	SPT	5	SANDY SILT (ML) Loose to medium dense, brown, slightly moist, fine sand	
6					▲ Solid auger drilling
7	3	SPT	6		▼ Mud rotary drilling No recovery
8	4	SPT	8	SANDY LEAN CLAY (CL) Stiff, dark brown, moist, fine sand	
9					
10	5	SPT	11	With gravel and medium sand	
11	6	SPT	13	CLAYEY SAND (SC) Medium dense, dark brown and rust, moist, fine sand	
12					
13					
14					
15					
16					
17					
18	7	Shelby	200 psi	FAT CLAY (CH) Medium stiff, dark gray, moist	
19					
20					
21	8	Shelby	240 psi		
22					
23					
24					
25					
26	9	Shelby	200 psi		
27				POORLY GRADED SAND (SP) Medium dense, olive brown, fine to medium sand	
28					End drilling 2:45; begin pull-out
29				Bottom of boring at 27.5 feet	
30					

PROJECT: UPPER GUADALUPE RIVER BORINGS San Jose, California	# Log of Boring No. GR-93-4

BORING LOCATION: Bank of Guadalupe River off Almaden Road south of Canoas Garden Road

DATE STARTED: 5/27/93	**DATE FINISHED:** 5/27/93	**NOTES:** Drilling equipment: Failing 1500 Drilling contractor: Pitcher Drilling - Ron Baker Drilling fluid: 2 sacks Super Gel-X and water Logged by: E. Chase
DRILLING METHOD: 4 7/8" solid auger/rotary wash - drag bit and rock bit		
HAMMER WEIGHT: 140 lbs	**DROP:** 30 inches	
SAMPLER: Shelby tube 3" x 36"/SPT		

DEPTH (feet)	SAMPLES			MATERIAL DESCRIPTION	REMARKS
	Sample No.	Sample	Blows/ Foot	Surface Elevation: 136.5	
1				RUBBLE FILL	Start drilling 10:50
2					
3					Loosing drilling fluid in fill, placed casing to 9 feet
4					
5					
6					▲ Solid auger drilling ▼ Mud rotary drilling
7					
8	1	SPT	7	SILTY GRAVEL (GM) Loose, reddish brown, moist to wet	
9					
10	2	SPT	13	SANDY LEAN CLAY (CL) Stiff, dark olive brown, moist, fine to medium sand	
11					
12					
13	3	SPT	12	Occasional gravel	
14					
15					
16					
17					
18	4	Shelby	320 psi	FAT CLAY (CH) Stiff, dark gray, moist	
19					
20					
21	5	Shelby	200 psi 300 psi		
22					
23				Bottom of boring at 22.5 feet	End drilling 12:30; begin pull-out; the drilling mud drained into the fill
24					
25					
26					
27					
28					
29					
30					

gt-1-88

Project No. 2374 | **Geomatrix Consultants**

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. GR-93-5

BORING LOCATION: On Guadalupe River bank behind Reese Property

DATE STARTED: 5/28/93

DATE FINISHED: 5/28/93

DRILLING METHOD: 4 7/8" solid auger/rotary wash - drag bit and rock bit

HAMMER WEIGHT: 140 lbs

DROP: 30 inches

SAMPLER: Shelby tube 3" x 36"/SPT

NOTES:
Drilling equipment: Failing 1500
Drilling contractor: Pitcher Drilling - Ron Baker
Drilling fluid: 2 sacks Super Gel-X and water
Logged by: E. Chase

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 162.0	REMARKS
1				SILTY SAND (SM) Very loose, dark brown, dry to slightly moist, fine sand	Start drilling 10:00
2					Casing placed to 5 feet
3					
4					▲ Solid auger drilling
5	1	SPT	2		▼ Mud rotary drilling
6	2	SPT	5		
7				Loose	
8	3	SPT	13		
9	4	SPT	22	POORLY GRADED GRAVEL with CLAY and SAND (GP – GC) Medium dense, dark olive brown, slightly moist, fine to coarse sand	
10					No recovery
11					
12					
13					Unable to push Shelby; gravels in the hole
14				SANDY LEAN CLAY (CL) Stiff, brown, moist, fine sand	
15	5	SPT	11		
16					
17					
18					
19	6	SPT	10		
20					
21					
22					
23	7	SPT	13		
24				Bottom of boring at 23.5 feet	End drilling 11:50; begin pull-out
25					
26					
27					
28					
29					
30					

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. GR-93-6

BORING LOCATION: Bank of Guadalupe River west of Wellington Square

DATE STARTED: 5/25/93	DATE FINISHED: 5/25/93	NOTES: Drilling equipment: CME-75 Drilling contractor: HEW Drilling - Jeff Ramanis Logged by: E. Chase
DRILLING METHOD: 8-inch hollow stem auger, finger cutting rock teeth bit		
HAMMER WEIGHT: 140 lbs	DROP: 30 inches	
SAMPLER: Shelby tube 3" x 36"/SPT		

DEPTH (feet)	SAMPLES Sample No.	SAMPLES Sample	SAMPLES Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 165.5	REMARKS
1				SILT with SAND (ML) Medium dense, dark brown, dry to slightly moist, fine sand	Start drilling 1:00
2					
3					
4				SILTY SAND (SM) Loose, brown, dry to moist, fine to medium sand, with some organics	
5	1	SPT	6		
6	2	SPT	7		
7					
8	3	SPT	7	SANDY LEAN CLAY (CL) Medium stiff, brown, slightly moist to moist, fine sand	
9	4	SPT	7	CLAYEY SAND (SC) Loose, brown, moist, fine sand	
10		Shelby	250 psi		
11	5	Shelby	350 psi	SANDY LEAN CLAY (CL) Medium stiff, brown, moist, fine sand	
12		Shelby	550 psi	LEAN CLAY (CL) Medium stiff, dark brown, dry to moist	
13					
14				Stiff	
15	6	SPT	18	SANDY LEAN CLAY (CL) Very stiff, yellowish brown, dry, fine sand	
16					
17		Shelby	400 psi		
18	7	Shelby	600 psi		
19		Shelby	1000 psi		
20	8	SPT	46	POORLY GRADED SAND with CLAY and GRAVEL (SP – SC) Very dense, olive brown, dry, fine to coarse sand	
21					
22					
23				SANDY LEAN CLAY (CL) Brown, stiff to very stiff, dry to moist, fine sand	
24					
25				▽	
26	9	Shelby	600 psi		
27					
28				POORLY GRADED SAND with CLAY and GRAVEL (SP – SC) Medium dense, brown, dry to moist, fine sand	
29					
30					

Water levels measured in temporary piezometer:

Date	Time	Depth
5/25	15:35	25.48
5/26	17:00	25.34
5/27	17:00	25.34

Water level in the river appears to be consistent with water levels in the piezometer

GI-1-88

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. GR-93-6 (cont'd.)

DEPTH (feet)	SAMPLES Sample No.	SAMPLES Sample	SAMPLES Blows/Foot	MATERIAL DESCRIPTION	REMARKS
	10	SPT	12	SANDY SILT (ML) Brown, medium dense, moist, fine sand	
31					
32	11	SPT	20	POORLY GRADED GRAVEL with SAND and CLAY (GP – GC) Medium dense, brown slightly moist, fine to coarse sand	
33	12	SPT	8	SANDY LEAN CLAY (CL) Stiff, olive brown, moist, fine sand	
34					
35	13	SPT	9		
36	14	SPT	15		
37				Very stiff	
38	15	SPT	15		
39	16	SPT	44	POORLY GRADED SAND with CLAY and GRAVEL (SP – SC) Dense, brown, moist, fine to coarse sand	
40				Bottom of boring at 40.0 feet	End drilling 3:00; begin pull-out and placement of piezometer
41					
42					
43					
44					
45					
46					
47					
48					
49					
50					
51					
52					
53					
54					
55					
56					
57					
58					
59					
60					
61					
62					
63					
64					

gr-2-88

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. GR-93-7

BORING LOCATION: West bank of Guadalupe River, north of intersection of Branham and Almaden Expressway

DATE STARTED: 5/26/93

DATE FINISHED: 5/26/93

DRILLING METHOD: 4 7/8" solid auger/rotary wash - drag bit and rock bit

HAMMER WEIGHT: 140 lbs

DROP: 30 inches

SAMPLER: Shelby tube 3" x 36"/SPT

NOTES:
Drilling equipment: Failing 1500
Drilling contractor: Pitcher Drilling - Ron Baker
Drilling fluid: 2 sacks Super Gel-X and water
Logged by: E. Chase

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 168.0	REMARKS
				LEAN CLAY (CL) Medium stiff, dark brown, dry	Start drilling 1:40
1					
2	1	Shelby	200 psi		↑ Solid auger drilling ↓ Mud rotary drilling
3					
4					
5				Gravels at 5.5 feet	Casing placed to 5 feet
6					
7	2	SPT	20	SANDY SILT (ML) Medium dense to dense, brown, moist, fine sand	
8					
9					
10					
11	3	Shelby	425 psi		
12					Shelby refusal
13					
14				POORLY GRADED GRAVEL with CLAY and SAND (GP – GC) Dense to very dense, dark olive brown, moist, fine to coarse sand	
15	4	SPT	51		Loosing drilling fluid
16	5	SPT	49		
17					
18	6	SPT	79		
19	7	SPT	49		
20					
21	8	SPT	45		
22	9	SPT	63		
23					
24	10	SPT	25		
25	11	SPT	15	LEAN CLAY (CL) Stiff, brown, moist	No recovery, pushed rock
26					
27	12	SPT	13		
28	13	SPT	13	POORLY GRADED SAND with CLAY (SP – SC) Loose, dark olive brown, moist, fine sand	No recovery
29					
30	14	SPT	7	Bottom of boring at 30.5 feet	End drilling 4:00; begin pull-out

B 7

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. CC-93-1

BORING LOCATION: South side of Canaos Creek, a few hundred feet west of Nightingale Drive

DATE STARTED: 5/27/93

DATE FINISHED: 5/27/93

DRILLING METHOD: 4 7/8" solid auger/rotary wash - drag bit and rock bit

HAMMER WEIGHT: 140 lbs

DROP: 30 inches

SAMPLER: Shelby tube 3" x 36"/SPT

NOTES:
Drilling equipment: Failing 1500
Drilling contractor: Pitcher Drilling - Ron Baker
Drilling fluid: 2 sacks Super Gel-X and water
Logged by: E. Chase

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 143.0	REMARKS
1				SANDY LEAN CLAY (CL) Medium stiff to stiff, brown, dry, fine sand	Start drilling 9:40
2	1	Shelby	250 psi		
3					Casing placed to 5 feet
4					
5					
6					▲ Solid auger drilling ▼ Mud rotary drilling
7	2	Shelby	250 psi		
8					
9				Dark brown	
10					
11	3	SPT	7	CLAYEY SAND (SC) Loose, brown, moist, fine to coarse sand, occasional gravel	
12					
13					
14					
15	4	Shelby	350 psi	SANDY LEAN CLAY (CL) Stiff to very stiff, brown, moist, occasional gravel	
16					
17					
18					
19					
20					
21	5	SPT	19		
22				Bottom of boring at 22.0 feet	End drilling 10:20; begin pull-out
23					
24					
25					
26					
27					
28					
29					
30					

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. CC-93-2

BORING LOCATION: South side of Canaos Creek, over 1000 feet west of Nightingale Drive

DATE STARTED: 5/27/93
DATE FINISHED: 5/27/93
DRILLING METHOD: 4 7/8" solid auger/rotary wash - drag bit and rock bit
HAMMER WEIGHT: 140 lbs
DROP: 30 inches
SAMPLER: Shelby tube 3" x 36"/SPT

NOTES:
Drilling equipment: Failing 1500
Drilling contractor: Pitcher Drilling - Ron Baker
Drilling fluid: 2 sacks Super Gel-X and water
Logged by: E. Chase

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 142.5	REMARKS
1	1	Shelby	250 psi	SILT (ML) Medium dense, brown, dry	Start drilling 7:50
2					
3					↑ Solid auger drilling
4	2	SPT	4	SANDY SILT (ML) Loose, brown, dry, fine sand	↓ Mud rotary drilling
5					
6	3	Shelby	200 psi	SANDY LEAN CLAY (CL) Medium stiff, brown, moist, fine sand	
7					Shelby refusal
8	4	SPT	13		
9					
10					
11	5	Shelby	200 psi		
12					
13					
14	6	Shelby	250 psi		
15					
16					
17					
18	7	SPT	73	POORLY GRADED GRAVEL with CLAY (GP – GC) Very dense, dark olive brown, moist	
19					
20					
21				SANDY LEAN CLAY (CL) Medium stiff, olive brown, fine sand	
22					
23					
24					
25					
26	8	Shelby	200 psi		
27					
28				Bottom of boring at 27.5 feet	End drilling 9:00; begin pull-out
29					
30					

gi-1-88

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. RC-93-1

BORING LOCATION: South side of Ross Creek

DATE STARTED: 5/25/93

DATE FINISHED: 5/25/93

DRILLING METHOD: 8-inch hollow stem auger, rock finger cutting teeth bit

HAMMER WEIGHT: 140 lbs

DROP: 30 inches

SAMPLER: Shelby tube 3" x 36"/SPT

NOTES:
Drilling equipment: CME-75
Drilling contractor: HEW Drilling - Jeff Ramanis
Logged by: E. Chase

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 168.5	REMARKS
1	1	Shelby	250 psi	SILTY SAND (SM) Medium dense, brown, dry, fine sand	Start drilling 8:30
2					
3	2	SPT	5		
4				SANDY SILT (ML) Loose, olive brown, dry to slightly moist, fine to medium sand	
5					
6					
7	3	Shelby	250 psi	SANDY LEAN CLAY (CL) Medium stiff, olive brown, slightly moist, fine to medium sand	
8			550 psi		
9					
10			250 psi		
11	4	Shelby			
12			400 psi		
13					
14					
15					
16					
17					
18	5	SPT	16	Very stiff	
19	6	SPT	25		
20	7	Shel		With some gravel	
21	8	SPT	30	POORLY GRADED GRAVEL with CLAY and SAND (GP – GC) Dense, dark olive brown, moist, fine to coarse sand ∇	Crushed Shelby tube
22					Water levels measured in temporary piezometer:
23					Date Time Depth
24					5/25 11:30 20.57 5/26 10:45 20.55
25					5/27 16:24 20.53
26	9	SPT	14		Water in creek due to recent rains. Water level appears to be consistent with the water levels in the piezometer.
27	10	SPT	15	SANDY LEAN CLAY with GRAVEL (CL) Stiff, olive brown, moist, fine to medium sand	
28					
29	11	SPT	14		
30		SPT			

Project No. 2374

Geomatrix Consultants

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. RC-93-1 (cont'd.)

DEPTH (feet)	SAMPLES Sample No.	SAMPLES Sample	SAMPLES Blows/ Foot	MATERIAL DESCRIPTION	REMARKS
	12	SPT	26	POORLY GRADED GRAVEL with CLAY and SAND (GP – GC) Medium dense, olive brown, moist, fine to coarse sand	
31					
32					
33					
34					
35	13	SPT	69	POORLY GRADED GRAVEL with SAND (GP) Very dense, dark olive brown, moist, fine to coarse sand	
36	14	SPT	51	CLAYEY SAND with GRAVEL (SC) Very dense, brown, moist, fine to coarse sand	
37					
38	15	SPT	64	POORLY GRADED GRAVEL with SAND (GP) Very dense, olive brown, moist, fine to coarse sand	
39	16	SPT	75	Increased coarse gravel	
40				Bottom of boring at 40.0 feet	End drilling 10:30; begin pull-out and placement of piezometer
41					
42					
43					
44					
45					
46					
47					
48					
49					
50					
51					
52					
53					
54					
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56					
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62					
63					
64					

gt-2-88

Project No. 2374

Geomatrix Consultants

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. RC-93-2

BORING LOCATION: South side of Ross Creek

DATE STARTED: 5/26/93

DATE FINISHED: 5/26/93

DRILLING METHOD: 4 7/8" solid auger/rotary wash - drag bit and rock bit

HAMMER WEIGHT: 140 lbs

DROP: 30 inches

SAMPLER: Shelby tube 3" x 36"/SPT/2 1/2-inch mod. Cal. drive sampler

NOTES:
Drilling equipment: Failing 1500
Drilling contractor: Pitcher Drilling - Ron Baker
Drilling fluid: 2 sacks Super Gel-X and water
Logged by: E. Chase

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 167.0	REMARKS
1				SANDY LEAN CLAY (CL) Stiff to very stiff, brown, dry to moist, fine sand	Start drilling 9:10
2	1	Shelby	300 psi		Casing placed to 5 feet
3					Shelby refusal
4	2	Cal	16		↑ Solid auger drilling
5					↓ Mud rotary drilling
6					
7					
8					
9	3	SPT	5	Medium stiff	
10	4	SPT	4		
11					
12	5	SPT	6	Occasional gravel	
13					
14					
15					
16					
17					
18					
19					
20					
21	6	Shel	400 psi	POORLY GRADED GRAVEL with CLAY and SAND (GP – GC) Very dense, olive brown, moist, fine to coarse sand	Crushed Shelby tube
22	7	SPT	78		Loosing drilling fluid
23					
24					
25					
26					
27					
28	8	SPT	50		
29	9	SPT	41	Dense	End drilling 11:00; begin pull-out
30				Bottom of boring at 30.0 feet	

gt-1-88

Project No. 2374

Geomatrix Consultants

B12

PROJECT: UPPER GUADALUPE RIVER BORINGS
San Jose, California

Log of Boring No. RC-93-3

BORING LOCATION: North side of Ross Creek

DATE STARTED: 5/26/93	DATE FINISHED: 5/26/93	NOTES: Drilling equipment: Failing 1500 Drilling contractor: Pitcher Drilling - Ron Baker Logged by: E. Chase
DRILLING METHOD: 4 7/8-inch solid auger, drag bit and rock bit		
HAMMER WEIGHT: 140 lbs	DROP: 30 inches	
SAMPLER: Shelby tube 3" x 36"/SPT		

DEPTH (feet)	SAMPLES Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION Surface Elevation: 170.0	REMARKS
1				SANDY LEAN CLAY (CL) Stiff, dark brown, dry, fine sand	Start drilling 11:45
2					
3	1	Shelby	450 psi		
4					
5					
6					
7	2	SPT	12	Brown	
8					
9	3	SPT	26	Very stiff, occasional gravel	
10				Increased gravel	
11					
12	4	SPT	45	Decreased gravel	
13					
14					
15					
16					
17	5	Shelby	300 psi		
18				Bottom of boring at 18.0 feet	Shelby refusal; end drilling 1:00; begin pull-out
19					
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					

B 13

ATTACHMENT C

LABORATORY TEST RESULTS

UPPER GUADALUPE RIVER PROJECT

U.S. ARMY CORPS OF ENGINEERS, SOUTH PACIFIC DIVISION LABORATORY, SAUSALITO, CALIFORNIA

SOIL TEST RESULT SUMMARY

PROJECT: Upper Guadulape River

DATE: October 1993

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	GRAVEL PERCENT FINER 6.0	3.0	1.5	3/4	3/8	SAND #4	#8	#16	#30	#50	#100	FINE #200	.005 MM	.001 MM	Specific Gravity	Liquid Limit	Plastic Index	Water Content %
121299	GR-93-2	1	3.5	5.0																	47	28	28
121401	GR-93-2	2	7.0	10.0																2.75	53	34	32
121402	GR-93-2	3	10.0	13.0	Fat Clay (CH)					100	99	99	99	99	99	98	98	68	27	2.76	62	41	39
121403	GR-93-2	4	20.0	23.0																2.74	53	34	30
121300	GR-93-3	1	3.0	4.5					100	99	94	87	81	79	76	64	50	16	10				32
121301	GR-93-3	2	4.5	6.0	Sandy Lean Clay (CL)					100	99	97	96	94	87	71	58	20	5	2.73	30	13	13
121302	GR-93-3	4	7.5	9.0	Fat Clay (CH)					100	99	99	99	99	99	97	95	43	12	2.71	52	34	28
121304	GR-93-3	6	10.5	12.0	Sandy Lean Clay (CL)					100	99	97	96	95	92	74	51				28	11	18
121399	GR-93-3	7	17.0	20.0																2.78	74	50	
121400	GR-93-3	8	20.0	23.0	Fat Clay (CH)												100	85	50	2.73	79	53	41
121404	GR-93-3	9	25.0	28.0																2.71	28	12	13
121305	GR-93-4	1	7.0	8.5					100	99	82	70	59	52	46	35	25						24
121306	GR-93-4	2	9.0	11.0																2.77	28	12	20
121307	GR-93-4	3	12.0	13.5								100	99	99	98	94	88	17	5	2.73			13
121405	GR-93-4	4	17.0	19.5																2.75	65	41	27
121406	GR-93-4	5	20.0	23.0	Fat Clay (CH)										100	99	99	84	48		72	48	30

U.S. ARMY CORPS OF ENGINEERS, SOUTH PACIFIC DIVISION LABORATORY, SAUSALITO, CALIFORNIA

SOIL TEST RESULT SUMMARY

PROJECT: Upper Guadulape River DATE: October 1993

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	GRAVEL PERCENT FINER					SAND							.005 MM	.001 MM	Specific Gravity	Liquid Limit	Plastic Index	Water Content %
						6.0	3.0	1.5	3/4	3/8	#4	#8	#16	#30	#50	#100	FINE #200						
121309	GR-93-5	2	5.5	7.0						100	99	98	97	96	88	62	38	13	5	2.73			20
121310	GR-93-5	3	7.0	8.5	Clayey Sand with Gravel (SC)				100	91	80	74	69	65	58	47	41	15	6	2.76	25	9	12
121311	GR-93-5	5	14.0	15.5																2.77	36	22	38
121312	GR-93-5	6	18.5	20.0	Sandy Lean Clay with Gravel (CL)				100	87	81	76	72	70	66	60	55	35	10		28	14	29
121313	GR-93-5	7	22.0	23.5	No Sample																		
121319	GR-93-6	2	5.5	7.0	Sandy Lean Clay (CL)								100	99	93	74	57	23	13		27	12	15
121318	GR-93-6	3	7.0	8.5	Lean Clay with Sand (CL)							100	99	97	93	84	74	21	18		33	15	19
121407	GR-93-6	5	10.0	13.0																2.77	45	27	23
121316	GR-93-6	6	14.0	15.5	Lean Clay with Sand (CL)								100	99	97	86	72	30	20		30	15	11
121411	GR-93-6	7	16.5	19.5																2.78	34	17	13
121315	GR-93-6	8	19.5	21.0	Clayey Gravel with Sand (GC)			100	88	76	57	44	36	30	23	17	14	3	2		29	15	40
121408	GR-93-6	9	25.0	28.0	Lean Clay with Sand (CL)							100	99	99	98	87	73	33	15	2.80	31	16	21
121314	GR-93-6	10	29.5	31.5	Sandy Lean Clay (CL)				100	97	91	88	87	86	84	77	68	19	9		29	13	24
121351	GR-93-6	11	31.0	32.5	Sandy Lean Clay (CL)					100	99	97	92	88	81	72	64	20	9		25	11	25
121324	GR-93-6	13	34.0	35.5	Lean Clay (CL)									100	99	98	94	41	23		38	25	29
121322	GR-93-6	15	37.0	38.5																	25	13	21

U.S. ARMY CORPS OF ENGINEERS, SOUTH PACIFIC DIVISION LABORATORY, SAUSALITO, CALIFORNIA

SOIL TEST RESULT SUMMARY

PROJECT: Upper Guadulape River DATE: October 1993

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	GRAVEL					PERCENT FINER		SAND					.005 MM	.001 MM	Specific Gravity	Liquid Limit	Plastic Index	Water Content %
						6.0	3.0	1.5	3/4	3/8	#4	#8	#16	#30	#50	#100	FINE #200						
121409	GR-93-7	1	1.0	4.0																	30	13	12
121361	GR-93-7	2	6.5	8.0	Sandy Lean Clay (CL)					100	98	95	93	90	85	74	63	23	10		26	11	22
121410	GR-93-7	3	10.0	12.0																	30	18	17
121360	GR-93-7	4	14.0	15.5	Clayey Sand with Gravel (SC)				100	93	72	60	52	43	32	25	23	14	7		26	13	11
121358	GR-93-7	6	17.0	18.5	Clayey Sand with Gravel (SC)				100	89	66	53	42	33	25	19	16	11	5		27	14	12
121355	GR-93-7	9	21.5	23.0	Clayey Sand with Gravel (SC)				100	86	69	56	46	38	28	22	18	10	5		28	16	14
121353	GR-93-7	12	26.0	27.5	Lean Clay with Sand (CL)					100	99	99	99	98	97	90	80	38	22		31	18	24
121352	GR-93-7	14	29.0	30.5	Sandy Lean Clay (CL)						100	99	98	98	94	80	66	25	10	2.76	29	14	25
121385	CC-93-1	1	1.0	4.0																	47	30	15
121386	CC-93-1	2	6.0	9.0																2.76	60	38	
121294	CC-93-1	3	10.5	12.0	Sandy Lean Clay (CL)						100	91	84	80	75	63	55	35	10		31	18	21
121389	CC-93-1	4	14.0	17.0																2.76	56	39	32
121295	CC-93-1	5	20.5	22.0	Lean Clay with Sand (CL)				100	97	96	95	94	93	91	83	77	40	10		36	19	14
121387	CC-93-2	1	1.0	3.0																	33	17	16
121388	CC-93-2	3	5.0	7.0																2.76	37	22	26
121298	CC-93-2	4	7.0	8.5																	30	15	21

U.S. ARMY CORPS OF ENGINEERS, SOUTH PACIFIC DIVISION LABORATORY, SAUSALITO, CALIFORNIA

SOIL TEST RESULT SUMMARY

PROJECT: Upper Guadulape River DATE: October 1993

Division Number	Hole Number	Field Sample No.	Depth or Elevation		Laboratory Descriptive Classification	GRAVEL PERCENT FINER					SAND						FINE	.005 MM	.001 MM	Specific Gravity	Liquid Limit	Plastic Index	Water Content %
			From	To		6.0	3.0	1.5	3/4	3/8	#4	#8	#16	#30	#50	#100	#200						
121390	CC-93-2	5	10.0	13.0																	48	29	25
121391	CC-93-2	6	13.0	16.0																	46	29	
121296	CC-93-2	7	17.5	19.0	Clayey Sand with Gravel (SC)				100	91	73	64	59	55	51	47	40	12	5	2.76	34	21	13
121392	CC-93-2	8	25.0	28.0																	30	16	20
121393	RC-93-1	1	0.0	2.5	Clayey Sand (SC)						100	88	78	69	58	48	41	20	9		26	12	11
121394	RC-93-1	3	6.0	9.0	Gravelly Lean Clay (CL)			100	99	88	74	74	74	73	73	65	64	33	16		38	21	23
121371	RC-93-1	6	18.5	20.0	Sandy Lean Clay (CL)					100	92	87	85	84	81	73	65	32	15		29	15	16
121370	RC-93-1	8	20.5	22.0	Clayey Sand with Gravel (SC)				100	91	79	67	61	57	53	47	40	24	14	2.74	37	22	11
121368	RC-93-1	10	26.5	28.0	Sandy Lean Clay (CL)				100	97	89	89	88	87	84	71	56	27	15		30	17	21
121365	RC-93-1	13	34.0	35.5				100	88	72	54	45	35	28	22	17	12	6	3				10
121362&3	RC-93-1	15&16	37.0	40.0					100	88	57	44	34	26	18	12	10	6	3				26
121396	RC-93-2	1	1.0	3.0																	26	12	25
121379	RC-93-2	4	9.5	11.0	Lean Clay (CL)						100	99	98	98	98	95	87	30	17	2.75	36	20	9
121376	RC-93-2	7	21.0	22.5	Clayey Sand with Gravel (SC)				100	88	71	56	46	39	31	25	23	14	5		32	19	9
121374&5	RC-93-2	8&9	27.0	30.0	Clayeay Gravel with Sand (GC)			100	79	70	55	45	35	27	22	18	15	7	3	2.77	39	26	11
121397	RC-93-3	1.0	2.0	5.0																	37	22	26

U.S. ARMY CORPS OF ENGINEERS, SOUTH PACIFIC DIVISION LABORATORY, SAUSALITO, CALIFORNIA

SOIL TEST RESULT SUMMARY

PROJECT: Upper Guadulape River DATE: October 1993

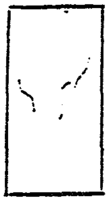
Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	GRAVEL PERCENT FINER 6.0	3.0	1.5	3/4	3/8	SAND #4	#8	#16	#30	#50	#100	FINE #200	.005 MM	.001 MM	UC TSF	Liquid Limit	Plastic Index	Water Content %
121382	RC-93-3	3	8.0	10.0	Sandy Lean Clay (CL)				100	98	89	84	81	79	75	69	64	35	18		30	17	9
121398	RC-93-3	5	16.0	18.0																2.74	30	15	13

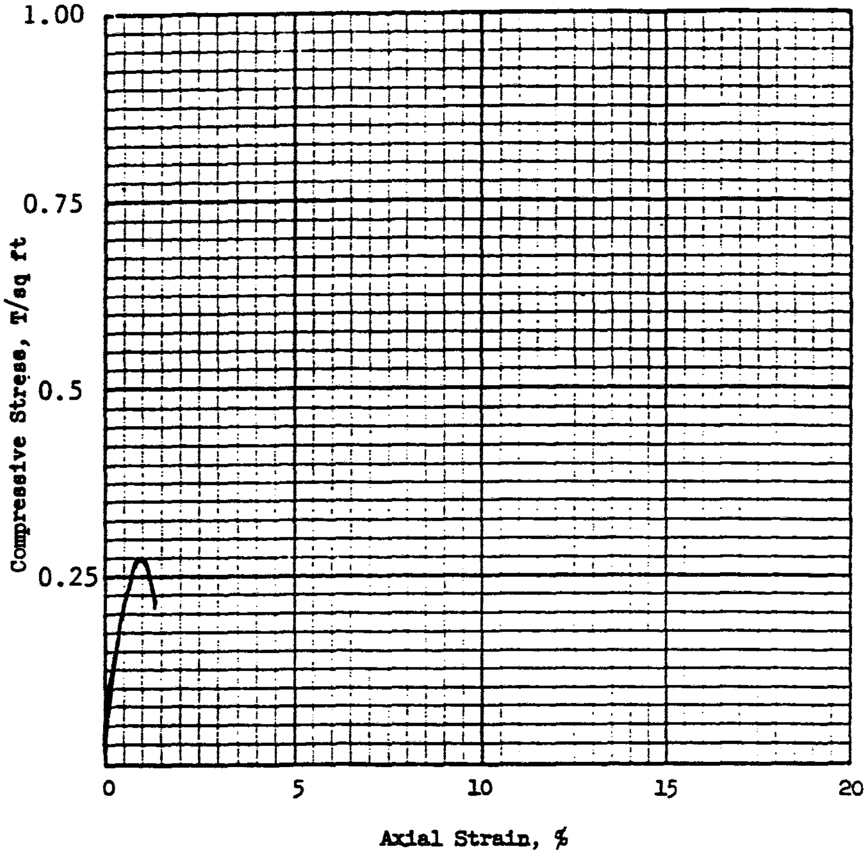
C5

UPPER GUADALUPE PROJECT (OCTOBER 1993)

Div. No.	Hole No.	Field Sample	UC Strength	
			TSF	PSI
121386	CC-93-1	2	0.27	3.75
389	CC-93-1	4	0.17	2.36
388	CC-93-2	3	0.17	2.4
390	CC-93-2	5	0.30	4.2
391	CC-93-2	6	1.01	14.0
401	GR-93-2	2	0.77	10.7
402	GR-93-2	3	0.86	11.9
405	GR-93-4	4	1.03	14.3
407	GR-93-6	5	1.53	21.53
409	GR-93-7	1	1.50	20.8
410	GR-93-7	3	0.35	4.9
399	GR-93-3	7	0.27	3.75
408	GR-93-6	9	0.88	12.2
394	RC-93-1	3	0.51	7.1

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.			121386			
Type of specimen			undist.			
Initial	Water content	w_o	29.6 %	%	%	%
	Void ratio	e_o	0.952			
	Saturation	S_o	85.5 %	%	%	%
	Dry density, lb/cu ft	γ_d	87.92			
Time to failure, min		t_f	1.66			
Unconfined compressive strength, T/sq ft		q_u	0.27			
Undrained shear strength, T/sq ft		s_u	0.14			
Sensitivity ratio		S_t	------			
Initial specimen diameter, cm		D_o	7.35			
Initial specimen height, cm		H_o	16.09			

Classification

LL	60	PL	38	PI	22	G_s	2.75

Remarks FS 2 @ 6' - 9'

Project Upper Guadalupe River

Santa Clara County, California

Area

Boring No. CC-93-1

Sample No. 121386

Depth El

Date October 1993

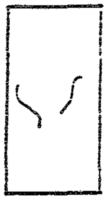
UNCONFINED COMPRESSION TEST REPORT

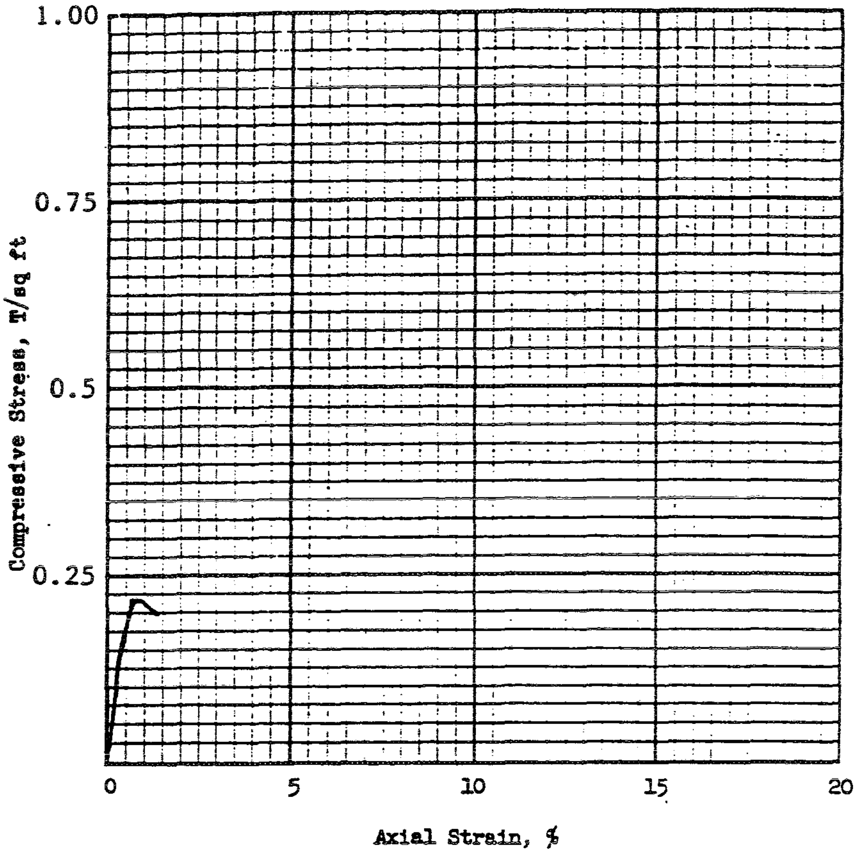
ENG FORM 1 JUN 65 3659

PLATE XI-2

C7

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.		121389			
Type of specimen		undist.			
Initial: Water content	w_o	29.7 %	%	%	%
Initial: Void ratio	e_o	0.913			
Initial: Saturation	S_o	89.6 %	%	%	%
Initial: Dry density, lb/cu ft	γ_d	89.72			
Time to failure, min	t_f	1.00			
Unconfined compressive strength, T/sq ft	q_u	0.17			
Undrained shear strength, T/sq ft	s_u	0.08			
Sensitivity ratio	S_t	----			
Initial specimen diameter, ~~in.~~ cm	D_o	7.33			
Initial specimen height, ~~in.~~ cm	H_o	16.98			

Classification

LL	56	PL	39	PI	17	G_s	2.75

Remarks FS 4 @ 14' - 17'

Project Upper Guadalupe River
Santa Clara County, California

Area

Boring No. CC-93-1

Sample No. 121389

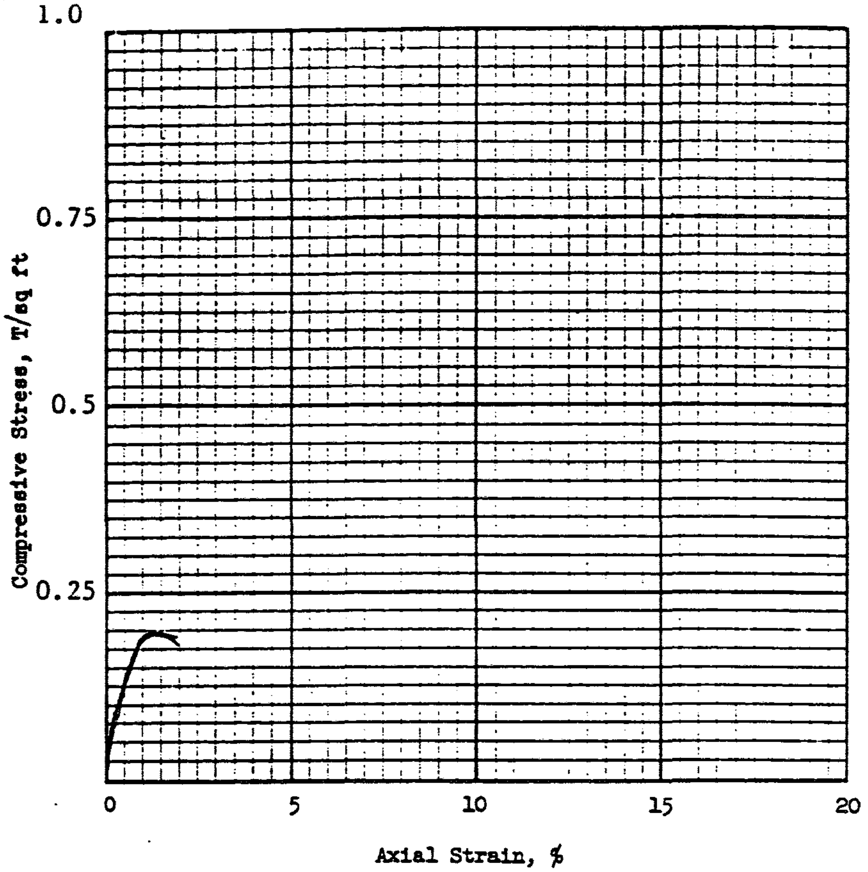
Depth
El

Date October 1993

UNCONFINED COMPRESSION TEST REPORT

ENG FORM 3659
1 JUN 65

Failure Sketches



☐ Controlled stress

☒ Controlled strain

Test No.			121388			
Type of specimen			undist.			
Initial	Water content	w_o	23.4 %	%	%	%
	Void ratio	e_o	0.815			
	Saturation	S_o	78.8 %	%	%	%
	Dry density, lb/cu ft	γ_d	94.52			
Time to failure, min		t_f	1.66			
Unconfined compressive strength, T/sq ft		q_u	0.17			
Undrained shear strength, T/sq ft		s_u	0.08			
Sensitivity ratio		S_t	----			
Initial specimen diameter, ~~in~~ cm		D_o	7.2			
Initial specimen height, ~~in~~ cm		H_o	15.29			

Classification

LL 37	PL 22	PI 15	G_s 2.75

Remarks FS 3 @ 5' - 7'

Project Upper Guadalupe River
Santa Clara County, California

Area

Boring No. CC 93-2

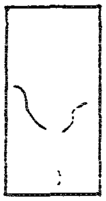
Sample No. 121388

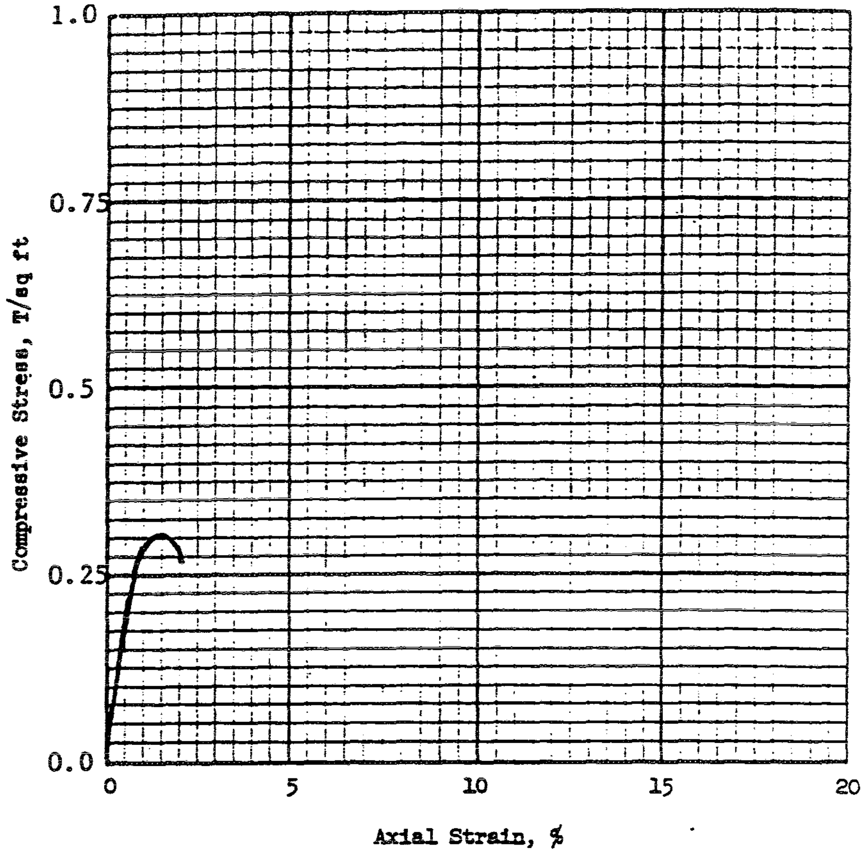
Depth El

Date October 1993

UNCONFINED COMPRESSION TEST REPORT

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.		121390			
Type of specimen		undist.			
Initial — **Water content**	w_o	39.6 %	%	%	%
Void ratio	e_o	1.151			
Saturation	S_o	94.6 %	%	%	%
Dry density, lb/cu ft	γ_d	79.79			
Time to failure, min	t_f	1.66			
Unconfined compressive strength, T/sq ft	q_u	0.30			
Undrained shear strength, T/sq ft	s_u	0.15			
Sensitivity ratio	S_t	----			
Initial specimen diameter, ~~in.~~ cm	D_o	7.21			
Initial specimen height, ~~in.~~ cm	H_o	16.32			

Classification

LL	48	**PL**	19	**PI**	29	G_s	2.75

Remarks FS 5 @ 10'

Project Upper Guadalupe River
Santa Clara County, California

Area

Boring No. CC-93-2 **Sample No.** 121390

Depth El **Date** October 1993

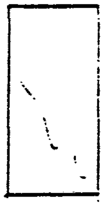
UNCONFINED COMPRESSION TEST REPORT

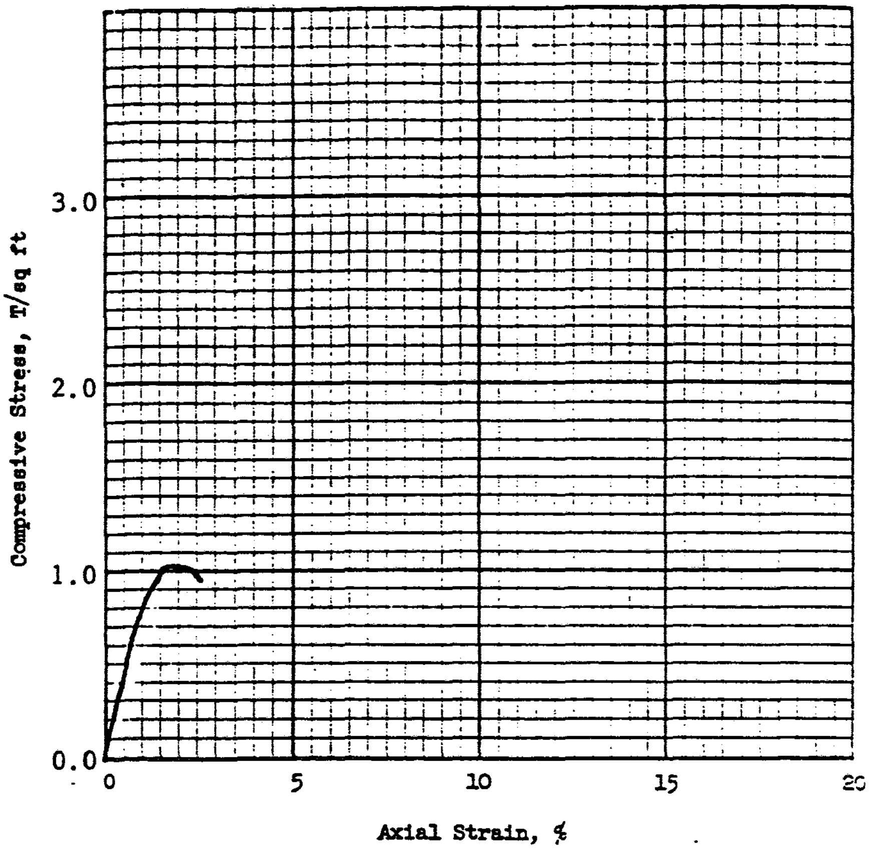
ENG FORM 1 JUN 65 3659

PLATE XI-2

C10

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.			121391			
Type of specimen			undist.			
Initial	**Water content**	w_o	18.3 %	%	%	%
	Void ratio	e_o	0.615			
	Saturation	S_o	81.6 %	%	%	%
	Dry density, lb/cu ft	γ_d	106.23			
Time to failure, min		t_f	3.66			
Unconfined compressive strength, T/sq ft		q_u	1.01			
Undrained shear strength, T/sq ft		s_u	0.50			
Sensitivity ratio		S_t	----			
Initial specimen diameter, ~~in~~ cm		D_o	7.34			
Initial specimen height, ~~in~~ cm		H_o	15.58			

Classification

LL	46	**PL**	17	**PI**	29	G_s	2.75

Remarks FS 6 @ 13'

Project Upper Guadalupe River
Santa Clara County, California

Area

Boring No. CC-93-2 **Sample No.** 121391

Depth El **Date** October 1993

UNCONFINED COMPRESSION TEST REPORT

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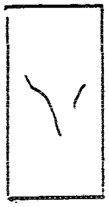
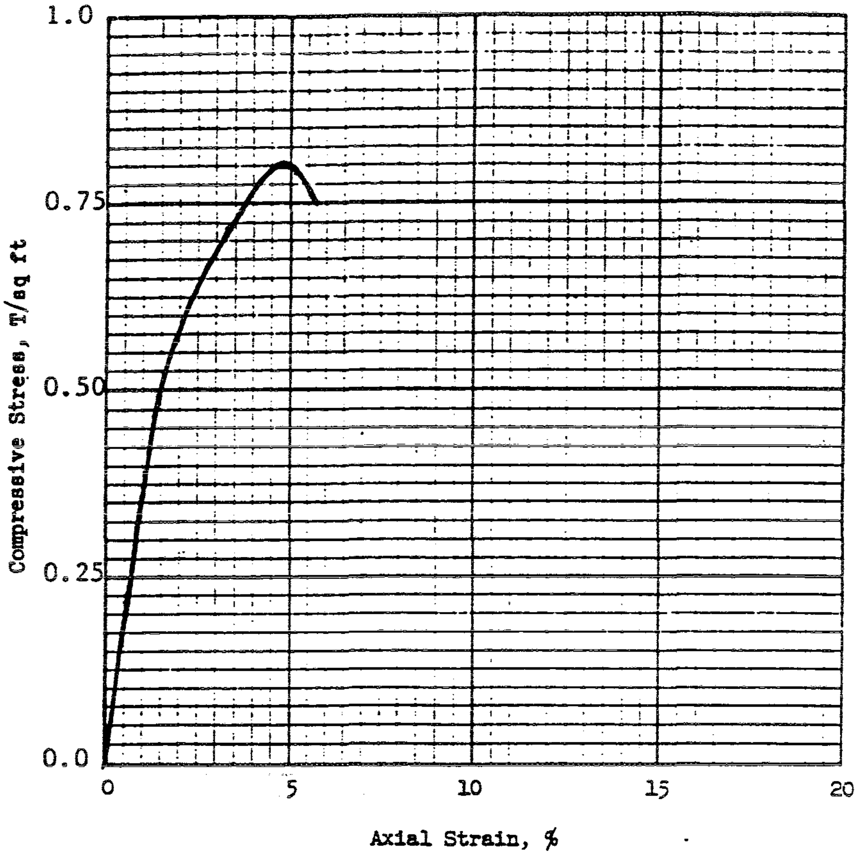
Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.		121401			
Type of specimen		undist.			
Initial – **Water content**	w_o	32.4 %	%	%	%
Initial – **Void ratio**	e_o	1.070			
Initial – **Saturation**	S_o	83.3 %	%	%	%
Initial – **Dry density, lb/cu ft**	γ_d	82.91			
Time to failure, min	t_f	6			
Unconfined compressive strength, T/sq ft	q_u	0.77			
Undrained shear strength, T/sq ft	s_u	0.39			
Sensitivity ratio	S_t	---			
Initial specimen diameter, ~~in~~ cm	D_o	7.2			
Initial specimen height, ~~in~~ cm	H_o	15.15			

Classification

LL	53 ~~47~~	**PL**	19	**PI**	34 ~~28~~	G_s	2.75

See soil test result summary sheet #1.

Remarks FS 2 @ 7'

Project Upper Guadalupe River
Santa Clara County, California

Area

Boring No. CR-93-2 **Sample No.** 121401

Depth El **Date** October 1993

UNCONFINED COMPRESSION TEST REPORT

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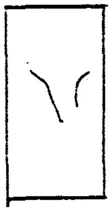
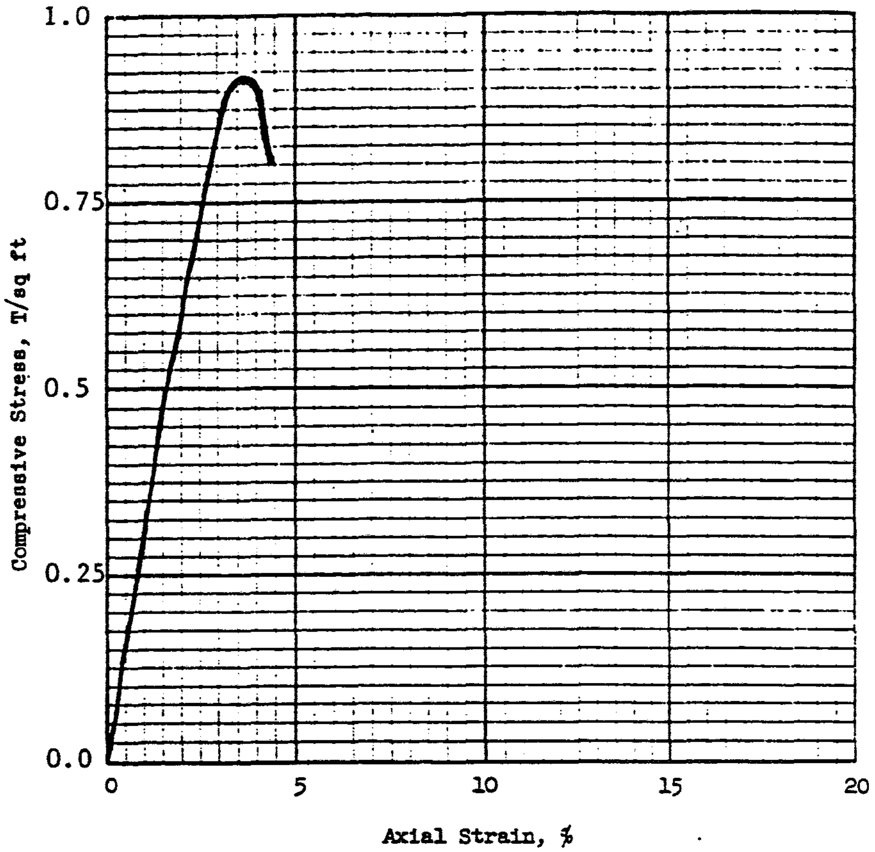
PLATE XI-2

C12

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.			121402			
Type of specimen			undist.			
Initial	Water content	w_o	39.0 %	%	%	%
	Void ratio	e_o	1.166			
	Saturation	S_o	92.1 %	%	%	%
	Dry density, lb/cu ft	γ_d	79.24			
Time to failure, min		t_f	6			
Unconfined compressive strength, T/sq ft		q_u	0.86			
Undrained shear strength, T/sq ft		s_u	0.43			
Sensitivity ratio		S_t	----			
Initial specimen diameter, ~~in~~ cm		D_o	7.24			
Initial specimen height, ~~in~~ cm		H_o	17.08			

Classification Fat Clay (CH)

LL	62	PL	21	PI	41	G_s	2.76

Remarks FS 3 @ 10'

Project Upper Guadalupe River
Santa Clara County, California

Area

Boring No. GR-93-2
Sample No. 121402

Depth
El

Date October 1993

UNCONFINED COMPRESSION TEST REPORT

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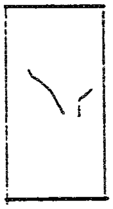
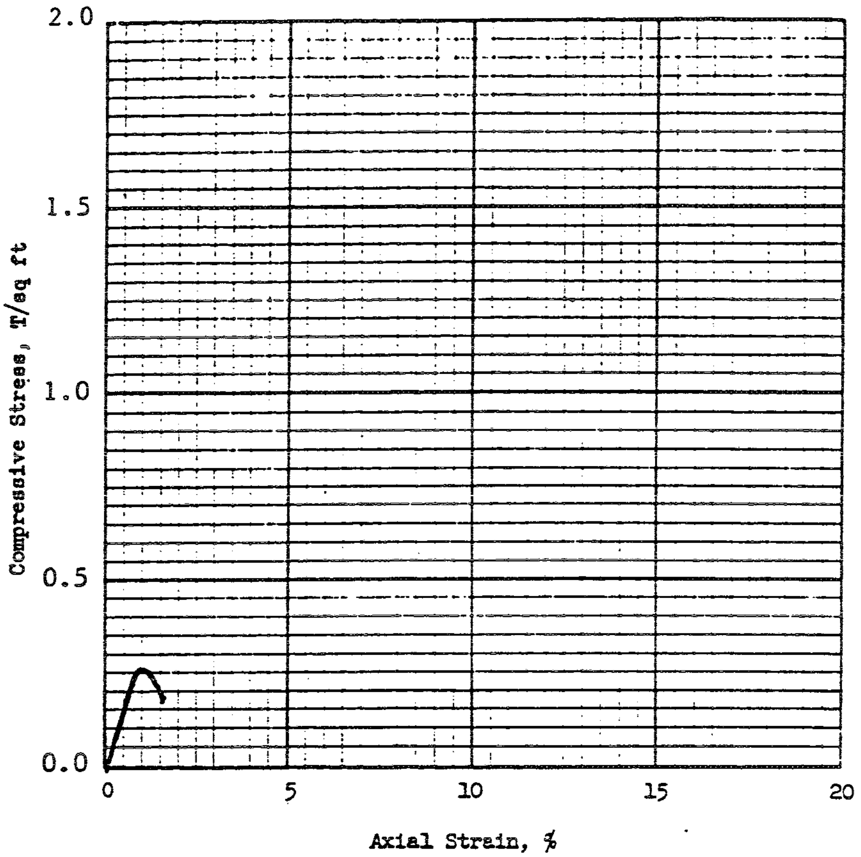
PLATE XI-2

C13

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.			121399			
Type of specimen			undist.			
Initial	**Water content**	w_o	38.8 %	%	%	%
	Void ratio	e_o	1.173			
	Saturation	S_o	90.8 %	%	%	%
	Dry density, lb/cu ft	γ_d	78.96			
Time to failure, min		t_f	1.33			
Unconfined compressive strength, T/sq ft		q_u	0.27			
Undrained shear strength, T/sq ft		s_u	0.14			
Sensitivity ratio		S_t	----			
Initial specimen diameter, ~~in.~~ cm		D_o	7.36			
Initial specimen height, ~~in.~~ cm		H_o	17.06			

Classification

LL	75	**PL**	24	**PI**	50	G_s	2.78

Remarks FS 7 @ 17'

Project	Upper Guadalupe River Santa Clara County, California
Area	
Boring No. CR-93-3	**Sample No.** 121399
Depth El	**Date** October 1993

UNCONFINED COMPRESSION TEST REPORT

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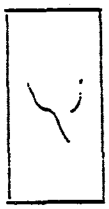
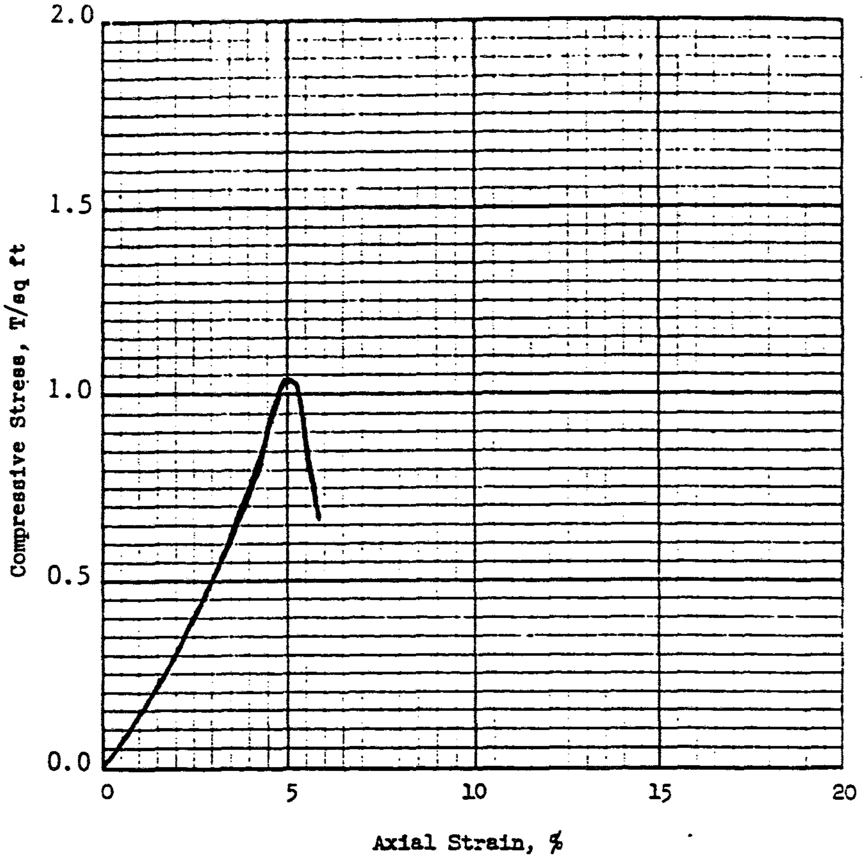
PLATE XI-2

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.			121405			
Type of specimen			undisturb.			
Initial	Water content	w_o	27.5 %	%	%	%
	Void ratio	e_o	1.001			
	Saturation	S_o	75.5 %	%	%	%
	Dry density, lb/cu ft	γ_d	85.74			
Time to failure, min		t_f	7			
Unconfined compressive strength, T/sq ft		q_u	1.03			
Undrained shear strength, T/sq ft		s_u	0.52			
Sensitivity ratio		S_t	---			
Initial specimen diameter, ~~in.~~ cm		D_o	7.23			
Initial specimen height, ~~in.~~ cm		H_o	17.19			

Classification

LL 65	PL 24	PI 41	G_s 2.75

Remarks FS 4 @ 17'

Project Upper Guadalupe River
Santa Clara County, California

Area

Boring No. GR-93-4 **Sample No.** 121405

Depth El **Date** October 1993

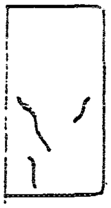
UNCONFINED COMPRESSION TEST REPORT

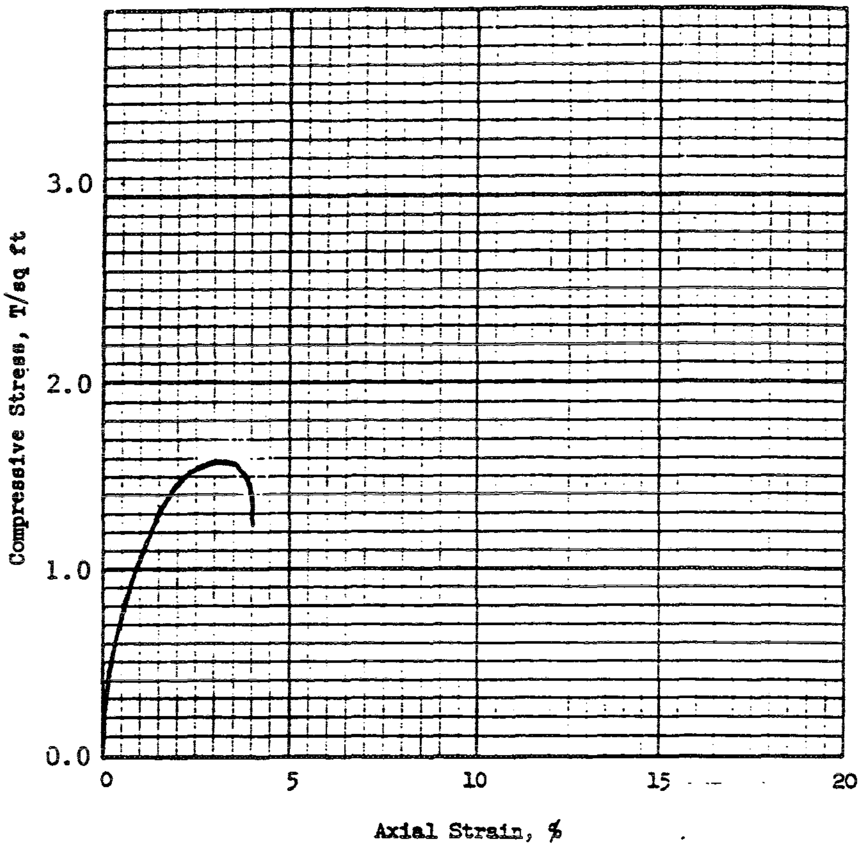
ENG FORM 1 JUN 65 3659

PLATE XI-2

C15

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.			121407			
Type of specimen			undist			
Initial	Water content	w_o	22.8 %	%	%	%
	Void ratio	e_o	0.741			
	Saturation	S_o	84.5 %	%	%	%
	Dry density, lb/cu ft	γ_d	98.54			
Time to failure, min		t_f	4			
Unconfined compressive strength, T/sq ft		q_u	1.53			
Undrained shear strength, T/sq ft		s_u	0.77			
Sensitivity ratio		S_t	----			
Initial specimen diameter, ~~in.~~ cm		D_o	7.27			
Initial specimen height, ~~in.~~ cm		H_o	16.76			

Classification

LL 45	PL 18	PI 27	G_s 2.77

Remarks FS 5 @ 10'

Project Upper Guadalupe River
Santa Clara County, California

Area

Boring No. GR-93-6
Sample No. 121407

Depth
El

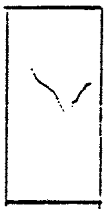
Date October 1993

UNCONFINED COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 3659

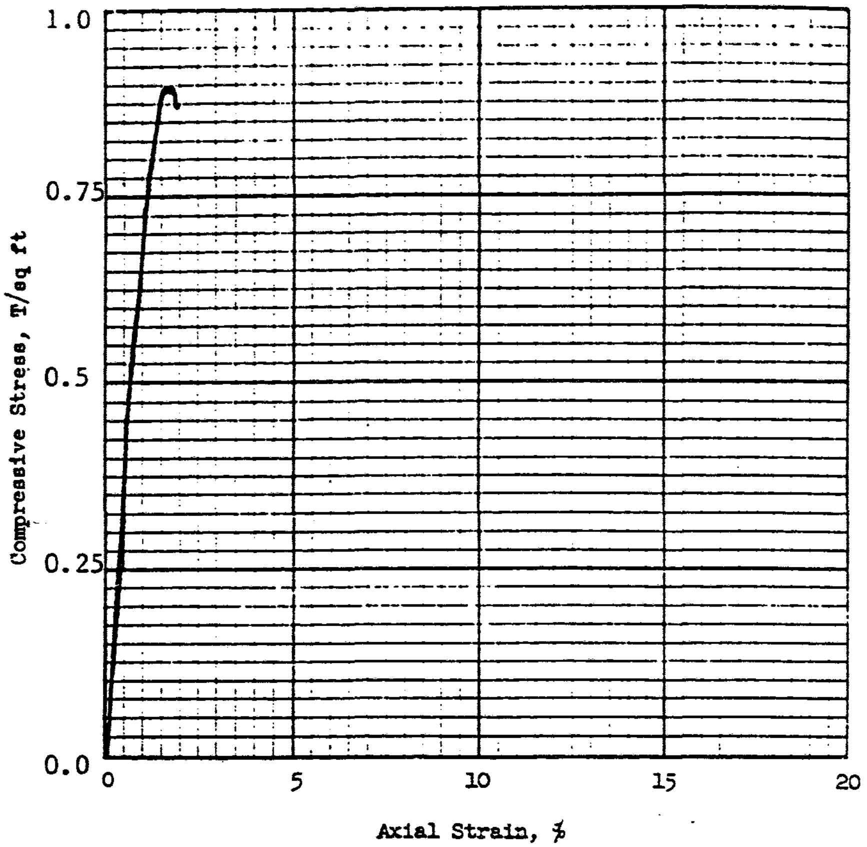
PLATE XI-2

Failure Sketches



☐ Controlled stress

☒ Controlled strain



Test No.		121408			
Type of specimen		undist.			
Initial: Water content	w_o	21.9 %	%	%	%
Initial: Void ratio	e_o	0.684			
Initial: Saturation	S_o	88.2 %	%	%	%
Initial: Dry density, lb/cu ft	γ_d	101.93			
Time to failure, min	t_f	3			
Unconfined compressive strength, T/sq ft	q_u	0.88			
Undrained shear strength, T/sq ft	s_u	0.44			
Sensitivity ratio	S_t	----			
Initial specimen diameter, ~~in~~ cm	D_o	7.37			
Initial specimen height, ~~in~~ cm	H_o	15.23			

Classification Lean Clay w/Sand (CL)

LL	31	PL	15	PI	16	G_s	2.80

Remarks FS 9 @ 25'

Project Upper Guadalupe River

Santa Clara County, California

Area

Boring No. GR-93-6

Sample No. 121408

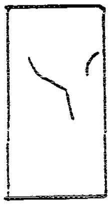
Depth El

Date October 1993

UNCONFINED COMPRESSION TEST REPORT

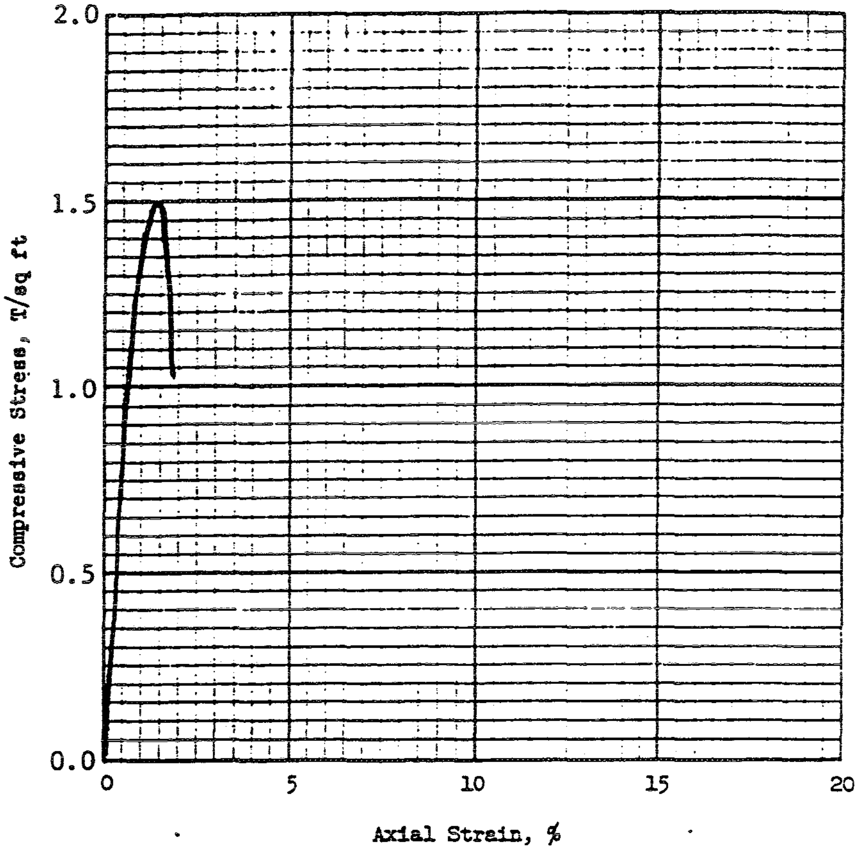
ENG FORM 1 JUN 65 3659

Failure Sketches



☐ Controlled stress

☒ Controlled strain



Test No.		121409			
Type of specimen		undist.			
Initial Water content	w_o	12.2 %	%	%	%
Initial Void ratio	e_o	0.863			
Initial Saturation	S_o	39.0 %	%	%	%
Initial Dry density, lb/cu ft	γ_d	92.10			
Time to failure, min	t_f	3			
Unconfined compressive strength, T/sq ft	q_u	1.50			
Undrained shear strength, T/sq ft	s_u	0.75			
Sensitivity ratio	S_t	----			
Initial specimen diameter, ~~in~~ cm	D_o	7.22			
Initial specimen height, ~~in~~ cm	H_o	16.97			

Classification

LL	30	PL	18	PI	13	G_s	2.75

Remarks FS 1 @ 1'

Project	Upper Guadalupe River Santa Clara County, California		
Area			
Boring No.	GR-93-7	**Sample No.**	121409
Depth El		**Date**	October 1993

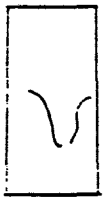
UNCONFINED COMPRESSION TEST REPORT

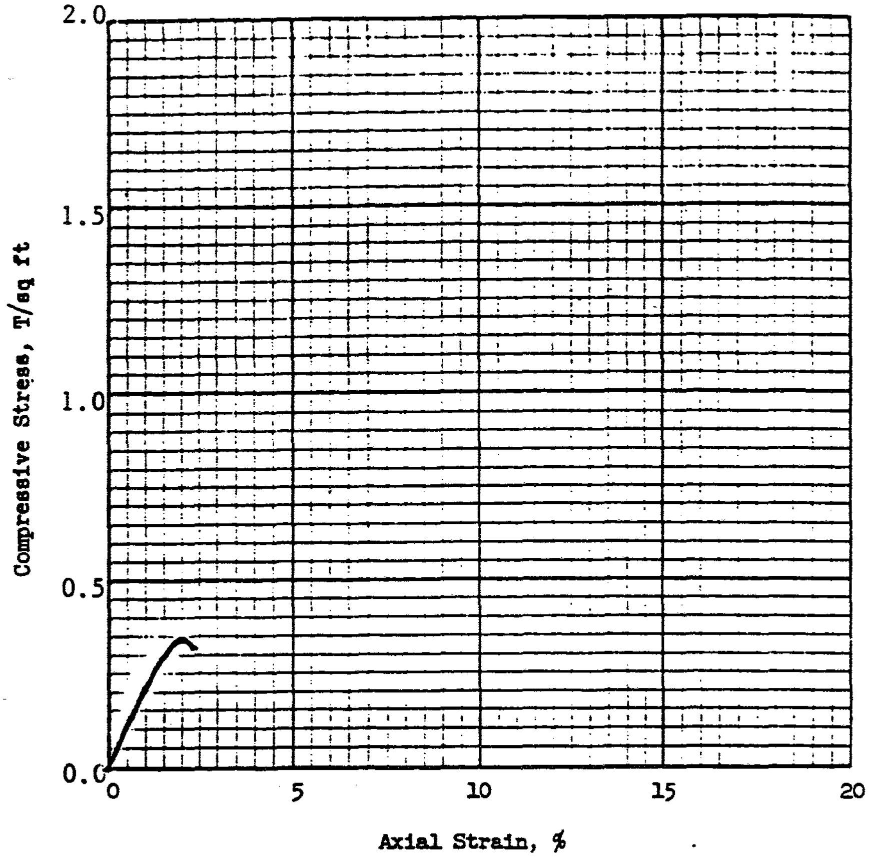
ENG FORM 1 JUN 65 3659

PLATE XI-2

C18

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.		121410			
Type of specimen		undist.			
Initial — Water content	w_o	20.2 %	%	%	%
Initial — Void ratio	e_o	0.830			
Initial — Saturation	S_o	66.9 %	%	%	%
Initial — Dry density, lb/cu ft	γ_d	93.77			
Time to failure, min	t_f	2.66			
Unconfined compressive strength, T/sq ft	q_u	0.35			
Undrained shear strength, T/sq ft	s_u	0.18			
Sensitivity ratio	S_t	----			
Initial specimen diameter, ~~in.~~ cm	D_o	7.24			
Initial specimen height, ~~in~~ cm	H_o	18.15			

Classification

LL	30	**PL**	13	**PI**	18	G_s	2.75

Remarks FS 3 @ 10'

Project Upper Guadalupe River
Santa Clare County, California

Area

Boring No. GR-93-7 **Sample No.** 121410

Depth El **Date** October 1993

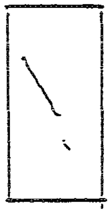
UNCONFINED COMPRESSION TEST REPORT

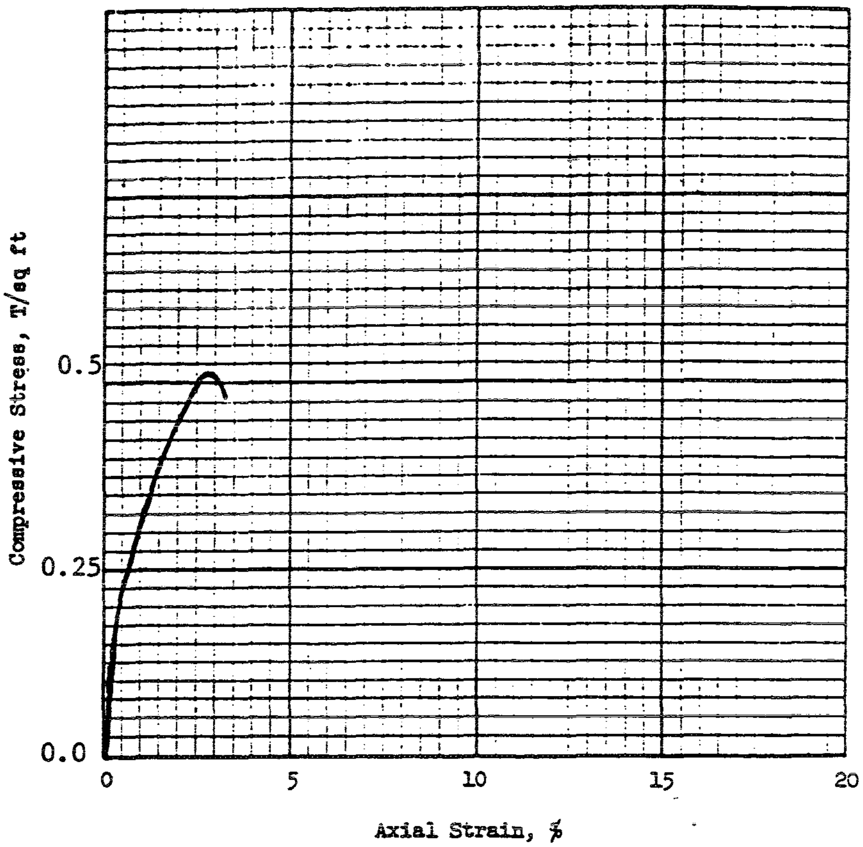
ENG FORM 1 JUN 65 3659

PLATE XI-2

C19

Failure Sketches





☐ Controlled stress

☒ Controlled strain

Test No.			121394			
Type of specimen			undist.			
Initial	Water content	w_o	23.8 %	%	%	%
	Void ratio	e_o	0.844			
	Saturation	S_o	77.7 %	%	%	%
	Dry density, lb/cu ft	γ_d	93.08			
Time to failure, min		t_f	4			
Unconfined compressive strength, T/sq ft		q_u	0.51			
Undrained shear strength, T/sq ft		s_u	0.26			
Sensitivity ratio		S_t	----			
Initial specimen diameter, ~~in.~~ cm		D_o	7.26			
Initial specimen height, ~~in.~~ cm		H_o	17.15			

Classification Gravelly Lean Clay (CL)

LL	38	PL	17	PI	21	G_s	2.75

Remarks FS 3 @ 6'

Project Upper Guadalupe River
Santa Clara County, California

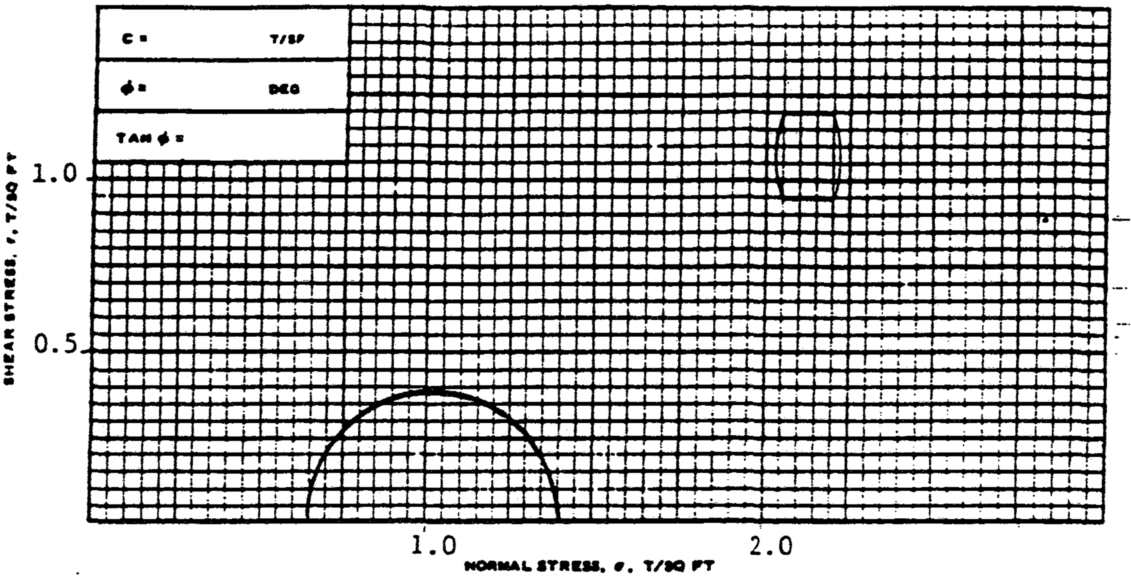
Area

Boring No. RC-93-1 **Sample No.** 121394

Depth El **Date**)ctober 1993

UNCONFINED COMPRESSION TEST REPORT

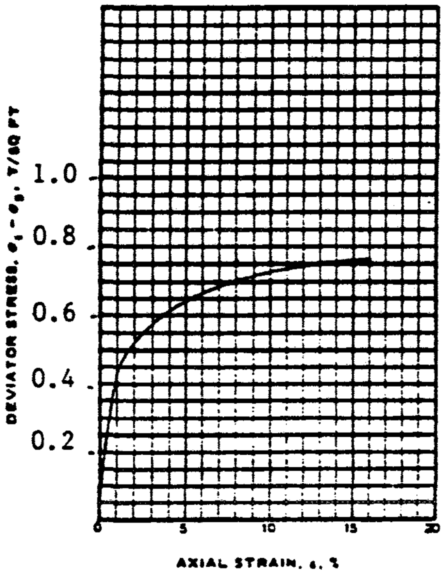
C20



C = T/SF

φ = DEG

TAN φ =



SPECIMEN NO.			1			
INITIAL	WATER CONTENT, %	w_o	43.8			
	DRY DENSITY LB/CU FT	γd_o	77.1			
	SATURATION, %	S_o	98.2			
	VOID RATIO	e_o	1.226			
BEFORE SHEAR	WATER CONTENT, %	w_c	49.0			
	DRY DENSITY LB/CU FT	γd_c	73.1			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	1.346			
	FINAL BACK PRESSURE, T/SQ FT	u_o	5.00			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.65			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.76			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_1	13.85			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	---			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.77			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 62	PL 21	PI 41	Gs *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: *GS- VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-2

SAMPLE NO. 121402

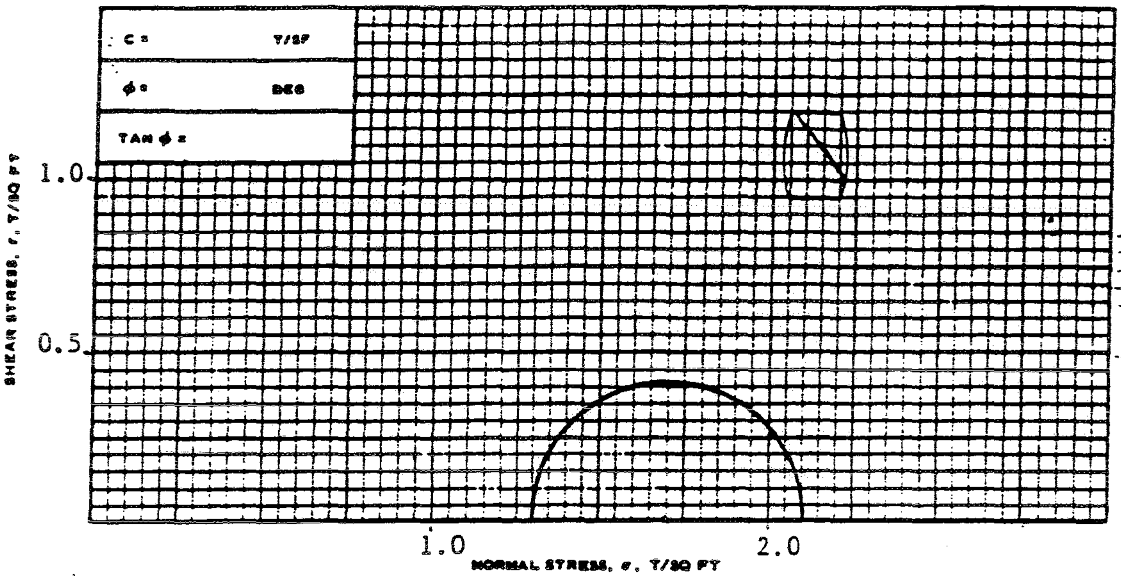
DEPTH/ELEV 10.0 - 13.0 FT.

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

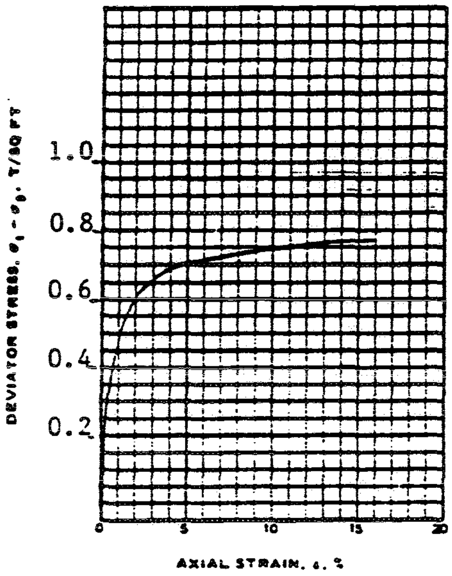
ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)



C = T/SF

φ = DEG

TAN φ =



SPECIMEN NO.			2			
INITIAL	WATER CONTENT, %	w_o	32.6			
	DRY DENSITY LB/CU FT	γd_o	83.7			
	SATURATION, %	S_o	85.2			
	VOID RATIO	e_o	1.051			
BEFORE SHEAR	WATER CONTENT, %	w_c	38.6			
	DRY DENSITY LB/CU FT	γd_c	83.3			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	1.061			
	FINAL BACK PRESSURE, T/SQ FT	u_o	5.00			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.30			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.83			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	12.75			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	---			
INITIAL DIAMETER, IN.		D_o	2.81			
INITIAL HEIGHT, IN.		H_o	6.76			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 53	PL 19	PI 34	Gs* 2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: *GS-VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-2

SAMPLE NO. 121403

DEPTH/ELEV 20.0 - 23.0 FT.

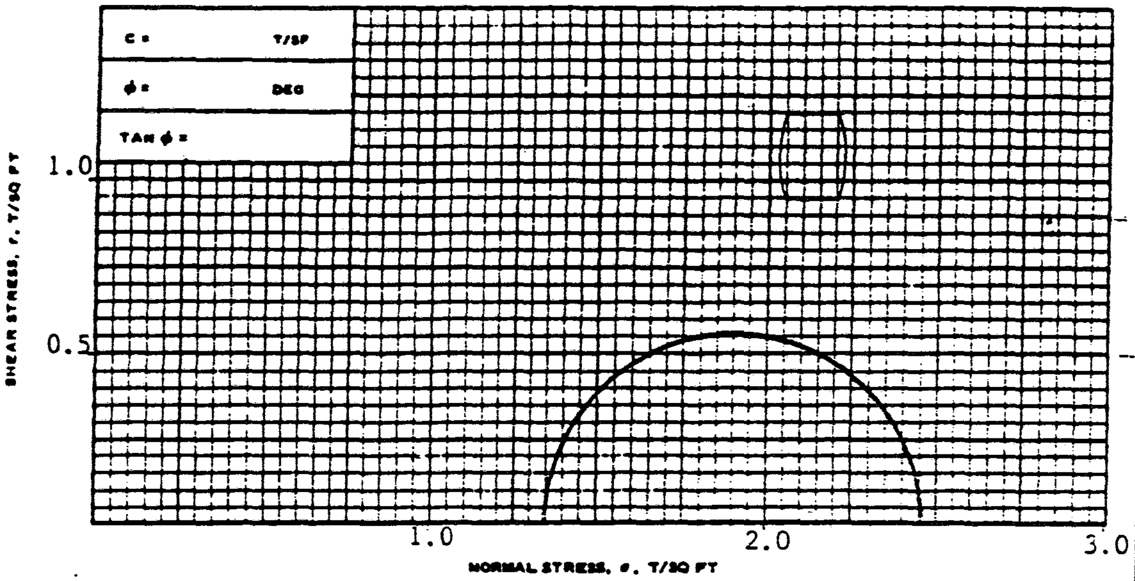
LABORATORY SPD

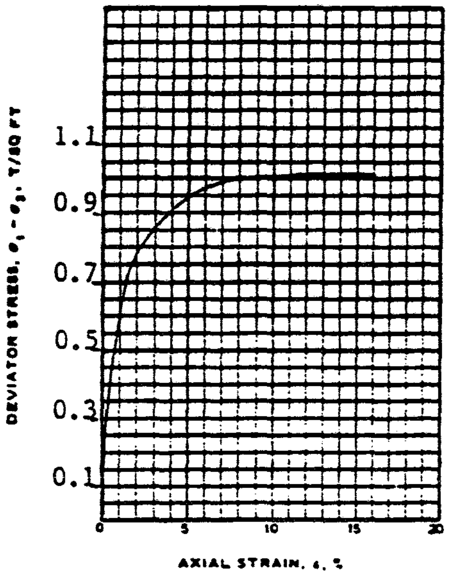
DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)
REV JUNE 1970

C22





SPECIMEN NO.		3			
INITIAL: WATER CONTENT, %	w_o	28.5			
INITIAL: DRY DENSITY LB/CU FT	γd_o	92.1			
INITIAL: SATURATION, %	S_o	90.7			
INITIAL: VOID RATIO	e_o	0.863			
BEFORE SHEAR: WATER CONTENT, %	w_c	32.4			
BEFORE SHEAR: DRY DENSITY LB/CU FT	γd_c	90.7			
BEFORE SHEAR: SATURATION, %	S_c	100.0			
BEFORE SHEAR: VOID RATIO	e_c	0.892			
BEFORE SHEAR: FINAL BACK PRESSURE, T/SQ FT	u_o	3.40			
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	1.36			
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{max}$	1.07			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN	t_f	13.75			
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ult}$	---			
INITIAL DIAMETER, IN.	D_o	2.80			
INITIAL HEIGHT, IN.	H_o	6.77			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 79 | PL 26 | PI 53 | G$_s$*2.75

TYPE OF SPECIMEN IN-SITU

TYPE OF TEST SAT. Q

REMARKS: *GS-VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-3

SAMPLE NO. 121400

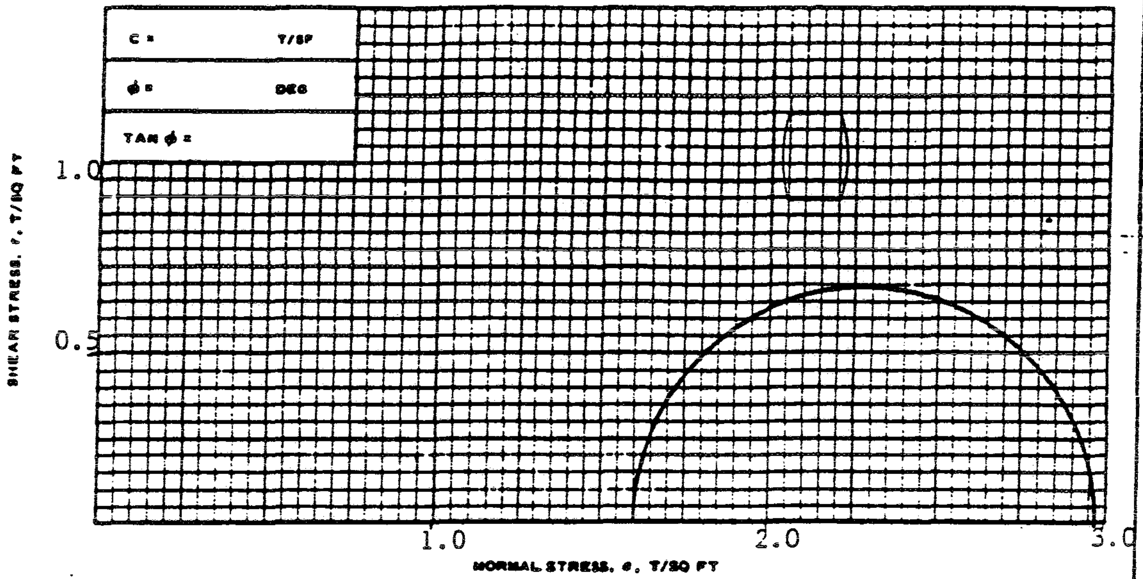
DEPTH/ELEV 20.0 - 23.0 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

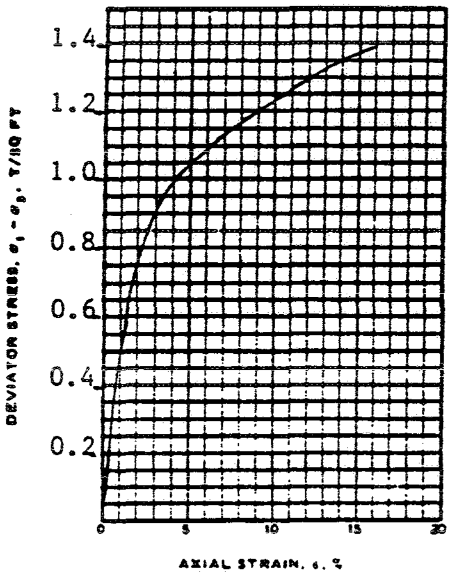
ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)



C = T/SF

φ = DEG

TAN φ =



SPECIMEN NO.			4			
INITIAL	WATER CONTENT, %	w_o	25.6			
	DRY DENSITY LB/CU FT	γd_o	95.9			
	SATURATION, %	S_o	89.2			
	VOID RATIO	e_o	0.789			
BEFORE SHEAR	WATER CONTENT, %	w_c	28.0			
	DRY DENSITY LB/CU FT	γd_c	96.9			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.770			
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.50			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.60			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	1.37			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	17.21			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	----			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.00			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 28	PL 16	PI 12	GS *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: * GS-VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-3

SAMPLE NO. 121404

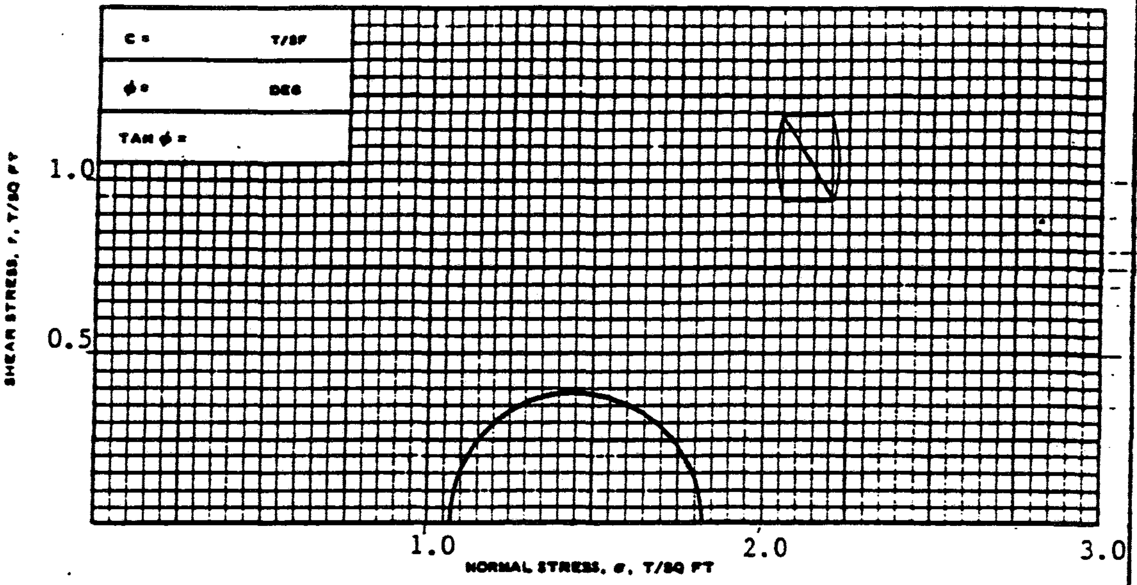
DEPTH/ELEV 25.0 - 28.0 FT

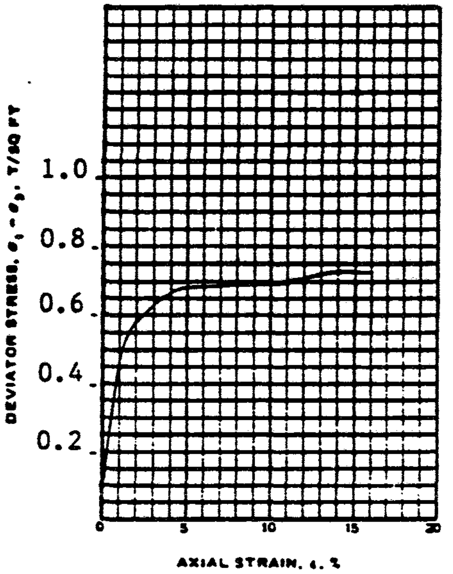
LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)





SPECIMEN NO.			5
INITIAL	WATER CONTENT, %	w_o	30.8
	DRY DENSITY LB/CU FT	γd_o	86.3
	SATURATION, %	S_o	85.6
	VOID RATIO	e_o	0.988
BEFORE SHEAR	WATER CONTENT, %	w_c	38.5
	DRY DENSITY LB/CU FT	γd_c	83.3
	SATURATION, %	S_c	100.0
	VOID RATIO	e_c	1.059
	FINAL BACK PRESSURE, T/SQ FT	u_o	5.04
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.08
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.73
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	11.45
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	————
INITIAL DIAMETER, IN.		D_o	2.80
INITIAL HEIGHT, IN.		H_o	6.32

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 65	PL 24	PI 41	G_s *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: GS-VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-4

SAMPLE NO. 121405

DEPTH/ELEV 17.0 - 19.5 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

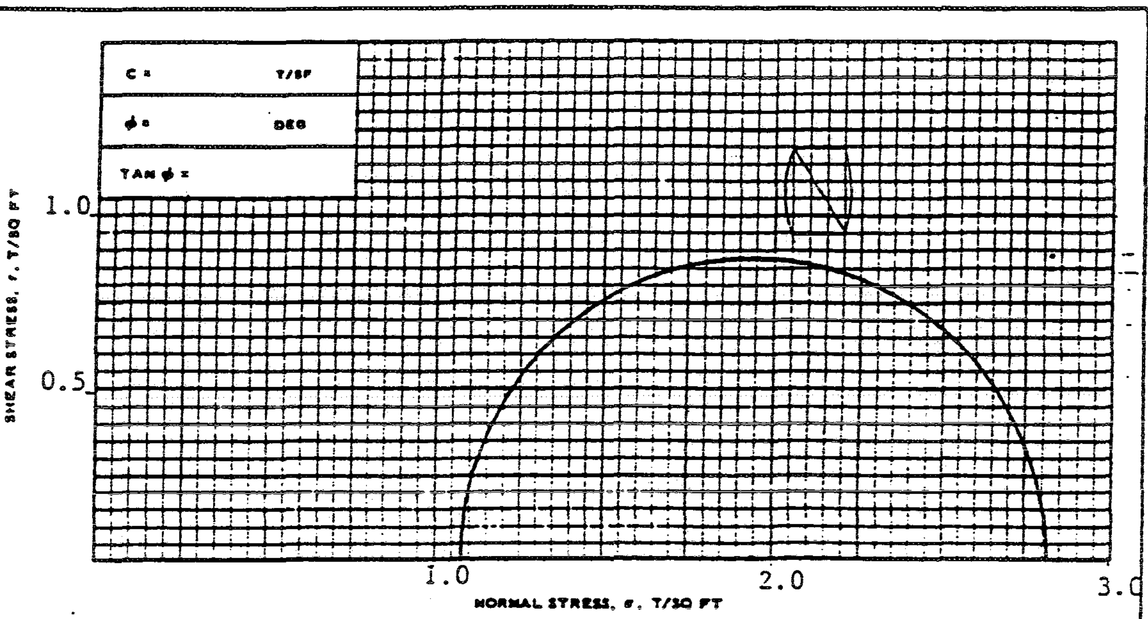
ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110-2-1906)*

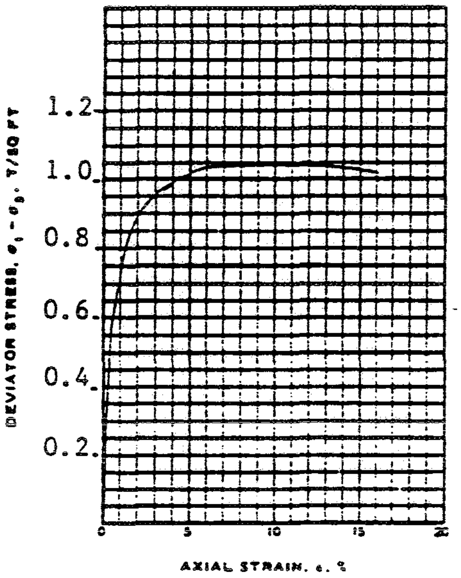
C25

C = T/SF

φ = DEG

TAN φ =





SPECIMEN NO.			6			
INITIAL	WATER CONTENT, %	w_o	30.5			
	DRY DENSITY LB/CU FT	γd_o	90.1			
	SATURATION, %	S_o	92.6			
	VOID RATIO	e_o	0.905			
BEFORE SHEAR	WATER CONTENT, %	w_c	35.0			
	DRY DENSITY LB/CU FT	γd_c	87.4			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.963			
	FINAL BACK PRESSURE, T/SQ FT	u_o	5.70			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.40			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	1.08			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	7.33			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	1.05			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.35			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 72 | PL 24 | PI 48 | GS *2.75 | TYPE OF SPECIMEN IN-SITU | TYPE OF TEST SAT. Q

REMARKS: GS-VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-4 | SAMPLE NO. 121406

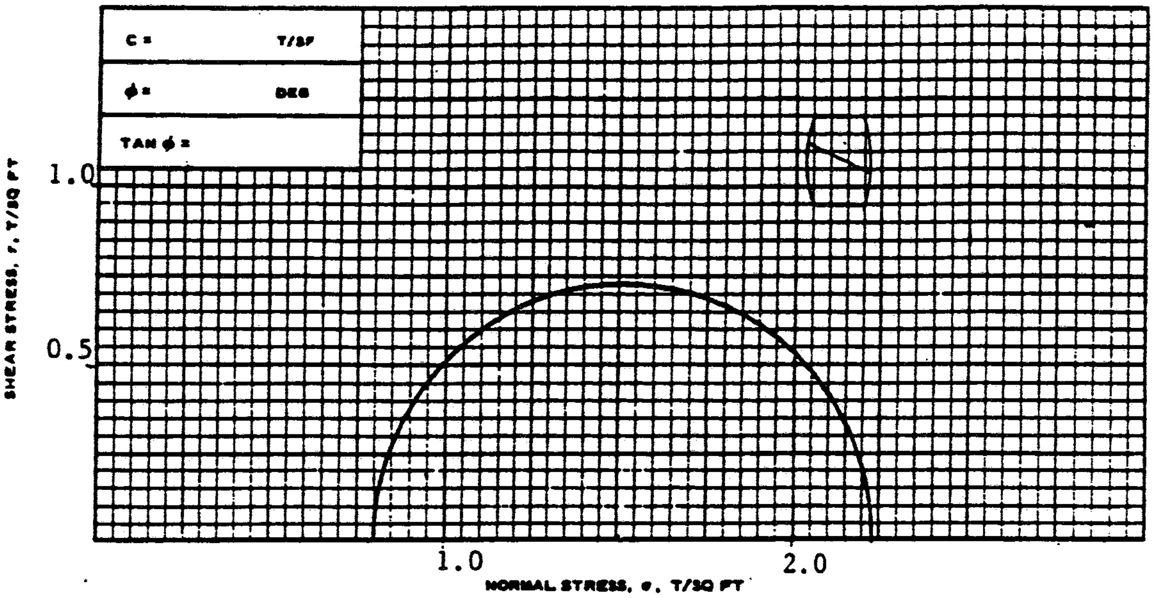
DEPTH/ELEV 20.0 - 23.0 FT

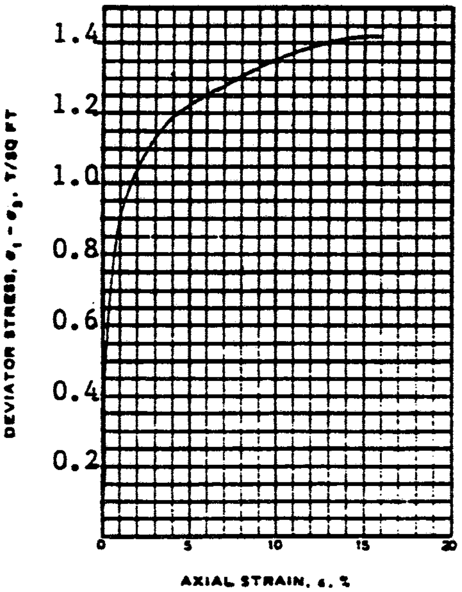
LABORATORY SPD | DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)

C =		T/SF
φ =		DEG
TAN φ =		





SPECIMEN NO.			7			
INITIAL	WATER CONTENT, %	w_o	19.0			
	DRY DENSITY LB/CU FT	γ_{d_o}	105.2			
	SATURATION, %	S_o	83.0			
	VOID RATIO	e_o	0.631			
BEFORE SHEAR	WATER CONTENT, %	w_c	24.0			
	DRY DENSITY LB/CU FT	γ_{d_c}	103.4			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.660			
	FINAL BACK PRESSURE, T/SQ FT	u_o	7.00			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.80			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	1.42			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	13.35			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	----			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.76			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 45	PL 18	PI 27	Gs *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: GS-VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-6

SAMPLE NO. 121407

DEPTH/ELEV 10.0 - 13.0 FT.

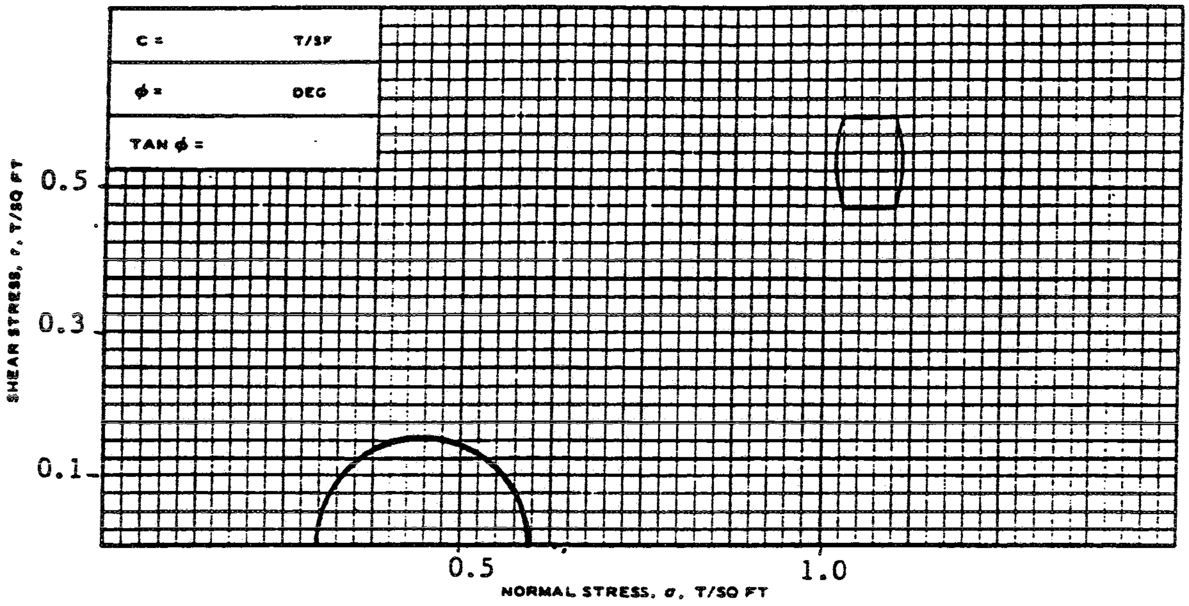
LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT *(EM 1110–2–1906)*

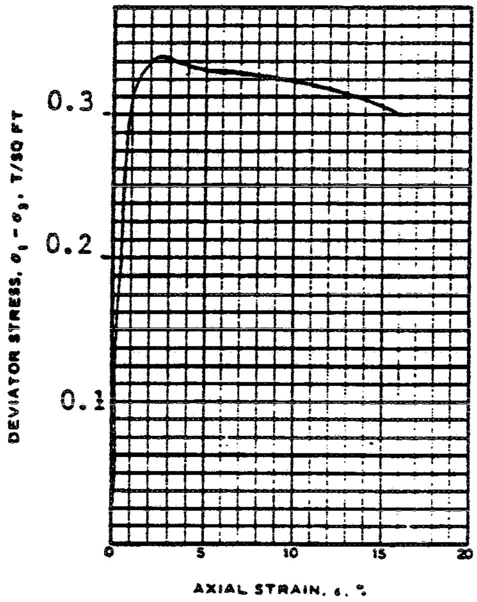
C27



C = T/SF

φ = DEG

TAN φ =



SPECIMEN NO.			8			
INITIAL	WATER CONTENT, %	w_o	12.5			
	DRY DENSITY LB/CU FT	γ_{d_o}	86.8	*		
	SATURATION, %	S_o	35.1			
	VOID RATIO	e_o	0.978			
BEFORE SHEAR	WATER CONTENT, %	w_c	30.4			
	DRY DENSITY LB/CU FT	γ_{d_c}	93.4	*		
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.837			
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.80			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.30			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.34			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	1.08			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	0.31			
INITIAL DIAMETER, IN.		D_o	2.81			
INITIAL HEIGHT, IN.		H_o	6.76			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 30 | PL 17 | PI 13 | GS *2.75 | TYPE OF SPECIMEN IN-SITU | TYPE OF TEST SAT. Q

REMARKS: GS-VALUE ASSUMED

* Variation in "initial" to "before shear" densities caused by consolidation under pressure saturation for requested "saturated Q-test".

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-7

SAMPLE NO. 121409

DEPTH/ELEV 1.0 - 4.0 FT

LABORATORY SPD

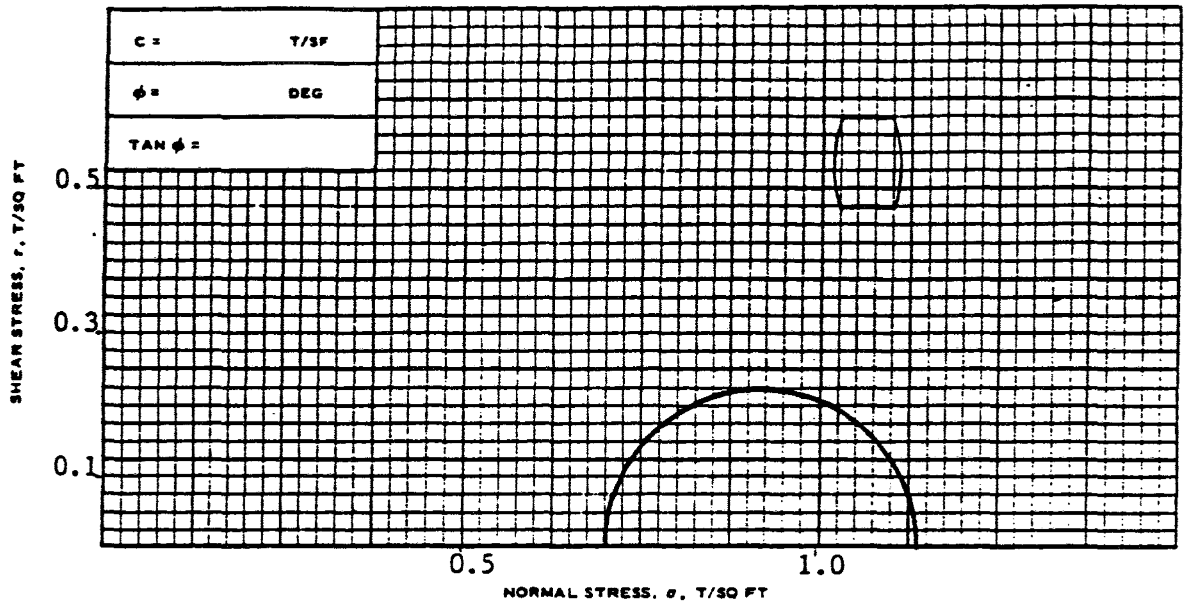
DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970

PREVIOUS EDITION IS OBSOLETE

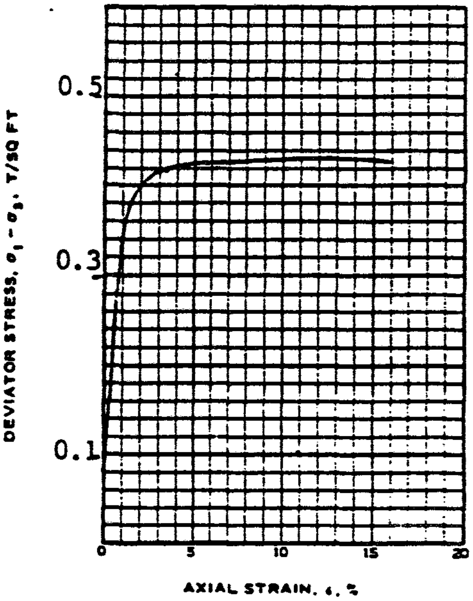
C28



C = T/SF

φ = DEG

TAN φ =



SPECIMEN NO.			9			
INITIAL	WATER CONTENT, %	w_0	20.9			
	DRY DENSITY LB/CU FT	γd_0	98.7			
	SATURATION, %	S_0	77.7			
	VOID RATIO	e_0	0.739			
BEFORE SHEAR	WATER CONTENT, %	w_c	25.1			
	DRY DENSITY LB/CU FT	γd_c	101.5			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.690			
	FINAL BACK PRESSURE, T/SQ FT	u_0	4.50			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.70			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.44			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	9.37			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	0.43			
INITIAL DIAMETER, IN.		D_0	2.83			
INITIAL HEIGHT, IN.		H_0	6.77			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 30	PL 13	PI 18	GS *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT, 0

REMARKS: GS-VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. GR-93-7

SAMPLE NO. 121410

DEPTH/ELEV 10.0 - 12.0 FT

LABORATORY SPD

DATE OCTOBER 1993

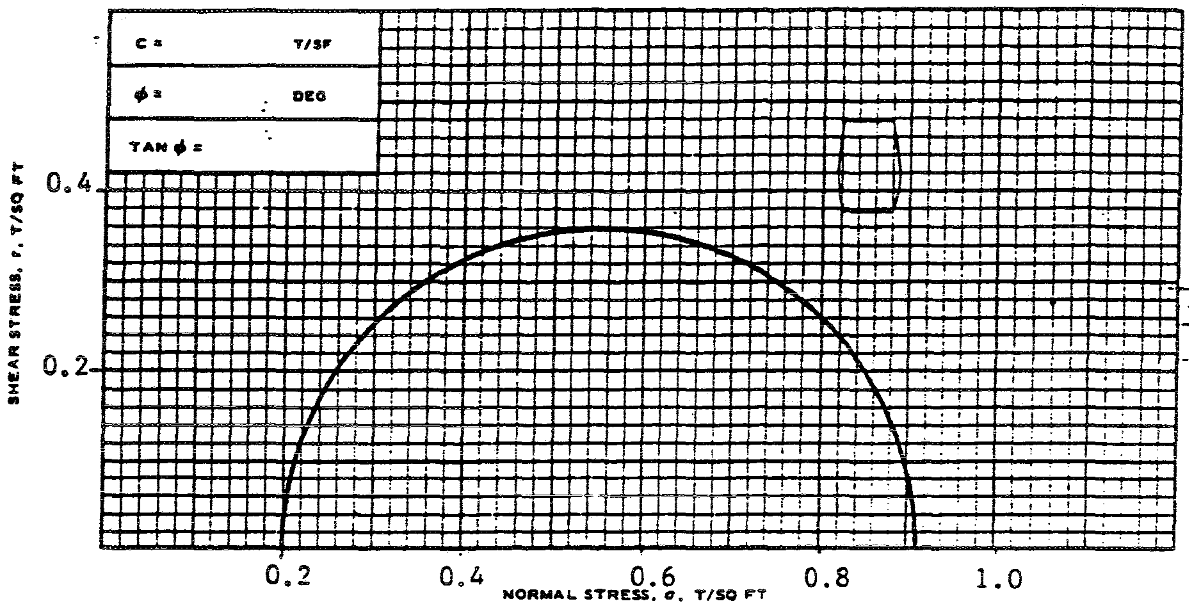
TRIAXIAL COMPRESSION TEST REPORT

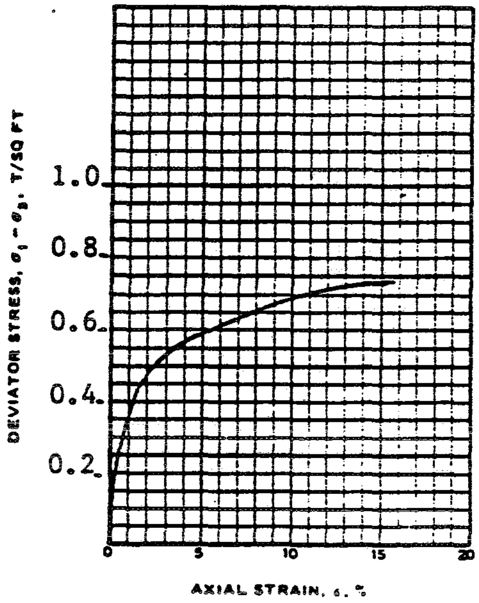
ENG FORM NO. 2089 PREVIOUS EDITION IS OBSOLETE
REV JUNE 1970

C = T/SF

φ = DEG

TAN φ =





SPECIMEN NO.		10			
INITIAL: WATER CONTENT, %	w_o	21.6			
INITIAL: DRY DENSITY LB/CU FT	γ_{d_o}	99.4			
INITIAL: SATURATION, %	S_o	81.6			
INITIAL: VOID RATIO	e_o	0.727			
BEFORE SHEAR: WATER CONTENT, %	w_c	28.1			
BEFORE SHEAR: DRY DENSITY LB/CU FT	γ_{d_c}	96.9			
BEFORE SHEAR: SATURATION, %	S_c	100.0			
BEFORE SHEAR: VOID RATIO	e_c	0.772			
BEFORE SHEAR: FINAL BACK PRESSURE, T/SQ FT	u_o	6.60			
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	0.20			
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{max}$	0.74			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN	t_f	13.68			
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ult}$	----			
INITIAL DIAMETER, IN.	D_o	2.81			
INITIAL HEIGHT, IN.	H_o	6.11			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 47	PL 17	PI 30	Gs *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: GS-VALUE-ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. CC-93-1

SAMPLE NO. 121385

DEPTH/ELEV 1.0 - 4.0 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

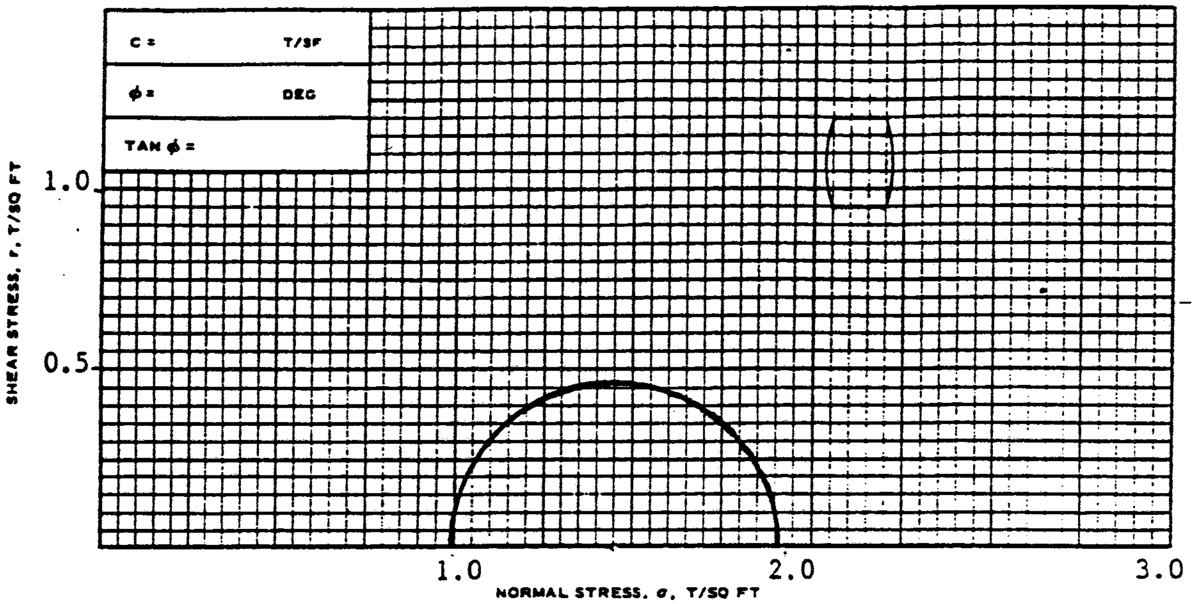
ENG FORM NO. 2089
REV JUNE 1970

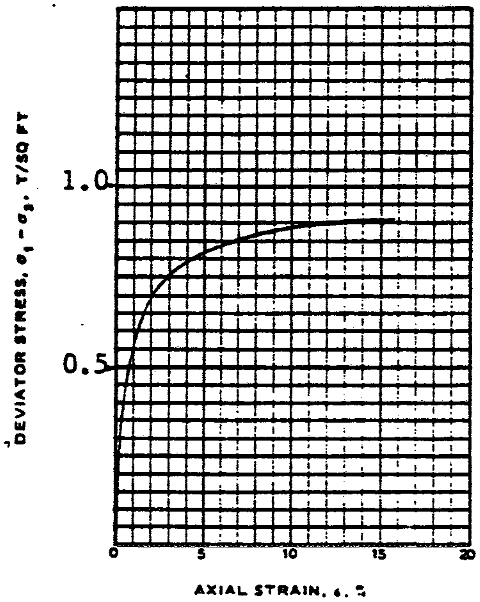
PREVIOUS EDITION IS OBSOLETE

C = T/SF

φ = DEG

TAN φ =





SPECIMEN NO.			11			
INITIAL	WATER CONTENT, %	w_o	29.5			
	DRY DENSITY LB/CU FT	γ_{d_o}	91.1			
	SATURATION, %	S_o	91.8			
	VOID RATIO	e_o	0.884			
BEFORE SHEAR	WATER CONTENT, %	w_c	32.3			
	DRY DENSITY LB/CU FT	γ_{d_c}	90.9			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.888			
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.30			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.90			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	15.58			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	----			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	5.99			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 56	PL 17	PI 39	Gs *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: GS-VALUE-ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. CC-93-1

SAMPLE NO. 121389

DEPTH/ELEV 14.0 - 17.0 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
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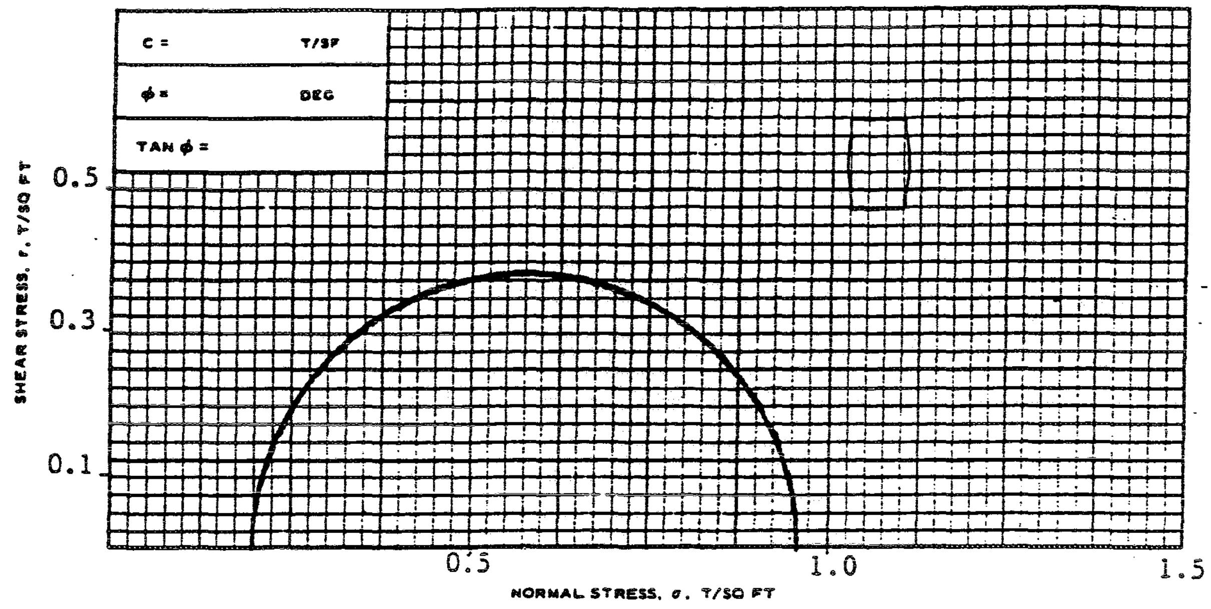
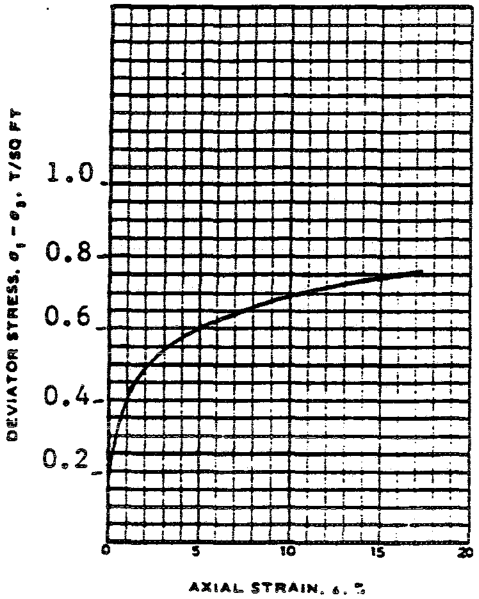


C = T/SF

φ = DEG

TAN φ =



SPECIMEN NO.			12			
INITIAL	WATER CONTENT, %	w_o	5.7			
	DRY DENSITY LB/CU FT	γ_{d_o}	118.2			
	SATURATION, %	S_o	34.5			
	VOID RATIO	e_o	0.452			
BEFORE SHEAR	WATER CONTENT, %	w_c	16.4			
	DRY DENSITY LB/CU FT	γ_{d_c}	118.2			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.451			
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.50			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.20			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.75			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	13.73			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	----			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.30			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 33 | PL 16 | PI 17 | G_s *2.75 | TYPE OF SPECIMEN IN-SITU | TYPE OF TEST SAT.Q

REMARKS:

PROJECT UPPER GUADALUPE RIVER

BORING NO. CC-93-2 | SAMPLE NO. 121387

DEPTH/ELEV 0.0 - 3.0 FT

LABORATORY SPD | DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
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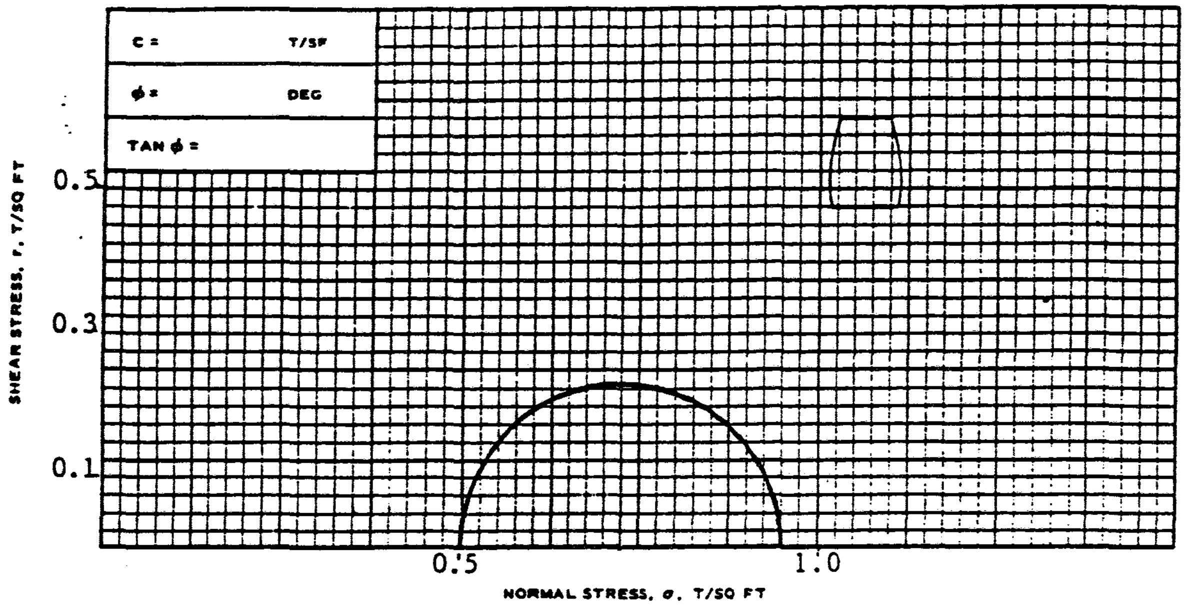
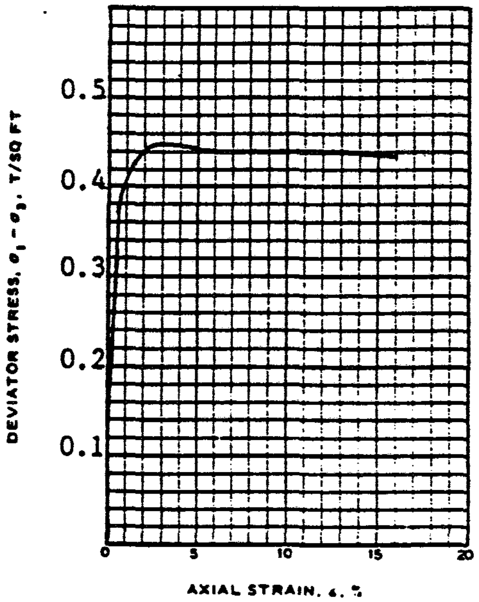
C32

c = T/SF

φ = DEG

TAN φ =





SPECIMEN NO.			13			
INITIAL	WATER CONTENT, %	w_o	21.5			
	DRY DENSITY LB/CU FT	γd_o	94.4			
	SATURATION, %	S_o	72.2			
	VOID RATIO	e_o	0.818			
BEFORE SHEAR	WATER CONTENT, %	w_c	26.5			
	DRY DENSITY LB/CU FT	γd_c	99.3			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.728			
	FINAL BACK PRESSURE, T/SQ FT	u_o	7.40			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.50			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.45			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	1.31			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	0.43			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.77			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 37	PL 15	PI 22	GS *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: GS-VALUE-ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. CC-93-2

SAMPLE NO. 121388

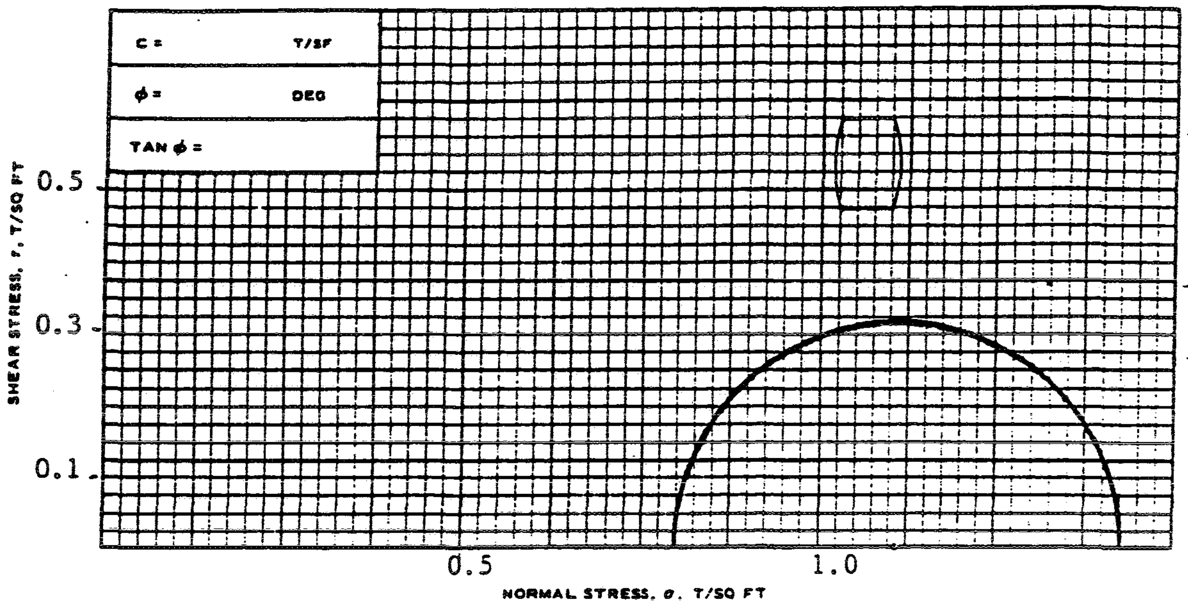
DEPTH/ELEV 5.0 - 7.0 FT

LABORATORY SPD

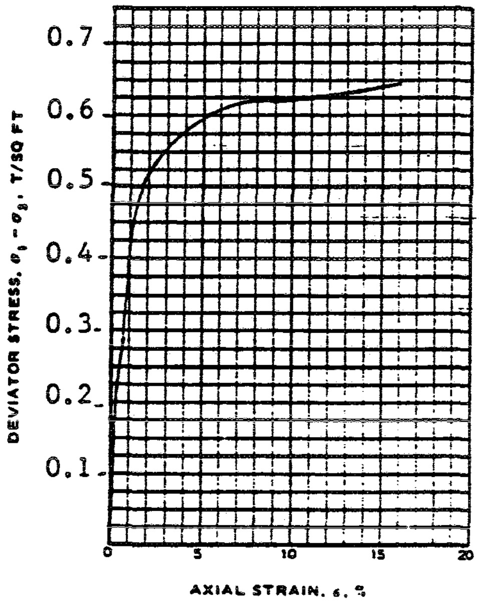
DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 PREVIOUS EDITION IS OBSOLETE
REV JUNE 1970



C =	T/SF
φ =	DEG
TAN φ =	



SPECIMEN NO.			14			
INITIAL	WATER CONTENT, %	w_o	28.5			
	DRY DENSITY LB/ CU FT	γd_o	88.3			
	SATURATION, %	S_o	83.2			
	VOID RATIO	e_o	0.944			
BEFORE SHEAR	WATER CONTENT, %	w_c	33.8			
	DRY DENSITY LB/CU FT	γd_c	89.0			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.929			
	FINAL BACK PRESSURE, T/SQ FT	u_o	5.00			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.80			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	0.63			
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	9.23			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	---			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.77			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 48	PL 19	PI 29	Gs *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: GS-VALUE-ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. CC-93-2

SAMPLE NO. 121390

DEPTH/ELEV 10.0 - 13.0 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

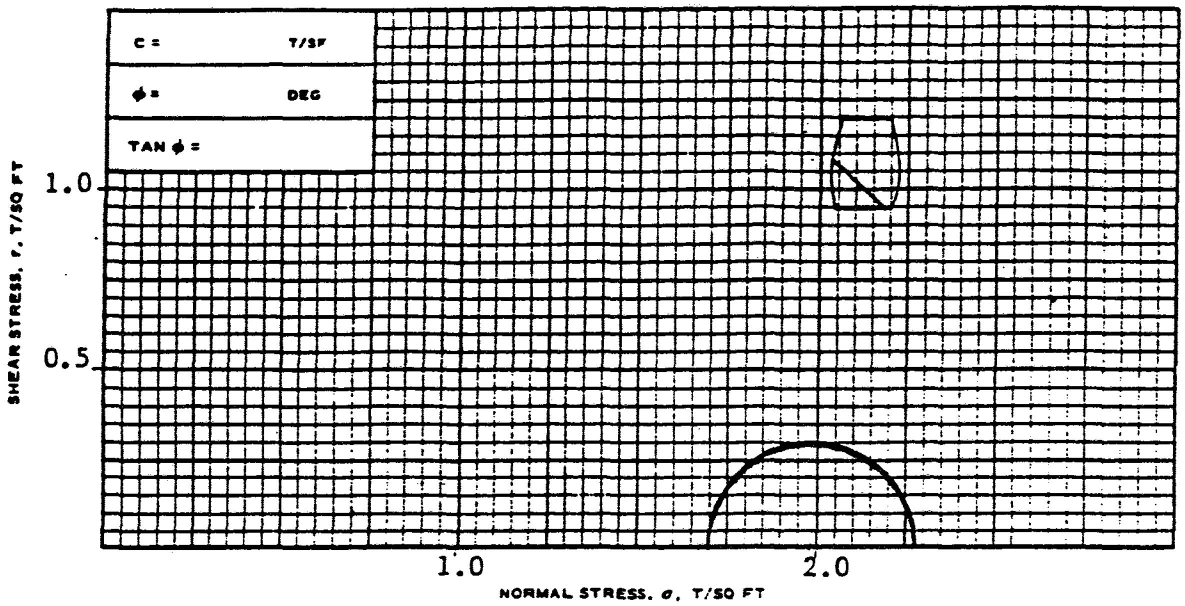
ENG FORM NO. 2089
REV JUNE 1970

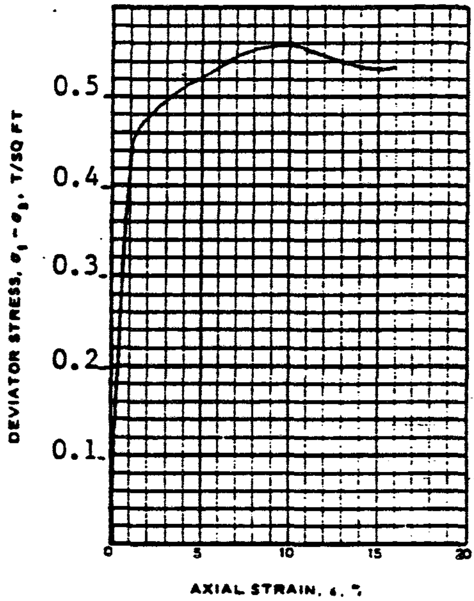
PREVIOUS EDITION IS OBSOLETE

C = T/SF

φ = DEG

TAN φ =





SPECIMEN NO.			15			
INITIAL	WATER CONTENT, %	w_o	20.3			
	DRY DENSITY LB/CU FT	γ_{d_o}	100.5			
	SATURATION, %	S_o	78.7			
	VOID RATIO	e_o	0.708			
BEFORE SHEAR	WATER CONTENT, %	w_c	24.1			
	DRY DENSITY LB/CU FT	γ_{d_c}	103.2			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.662			
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.70			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.70			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.56			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	7.95			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	0.53			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.77			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 30	PL 14	PI 16	GS *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. O

REMARKS: GS-VALUE-ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. CC-93-2

SAMPLE NO. 121392

DEPTH/ELEV 25.0 - 28.0 FT

LABORATORY SPD

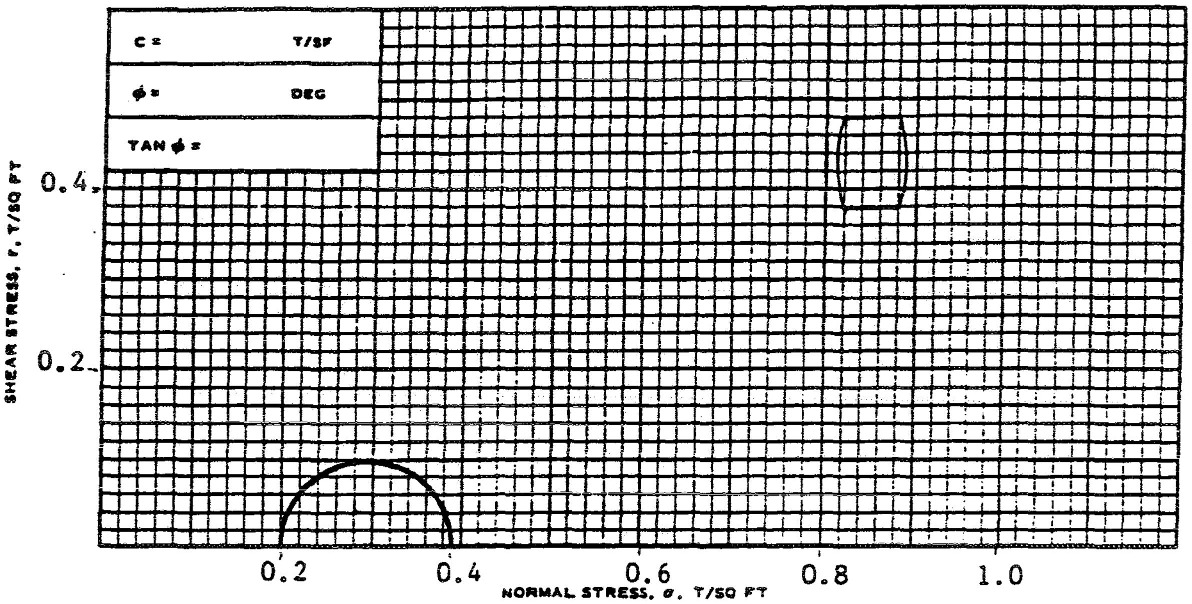
DATE OCTOBER 1993

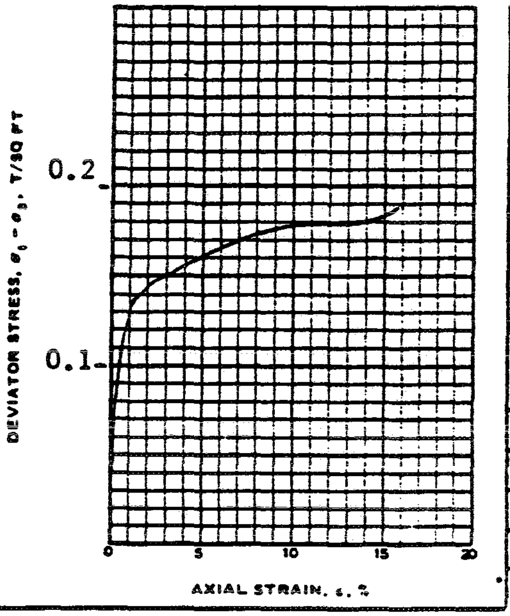
TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970

PREVIOUS EDITION IS OBSOLETE

C =	T/SF
φ =	DEG
TAN φ =	





SPECIMEN NO.			16			
INITIAL	WATER CONTENT, %	w_o	10.6			
	DRY DENSITY LB/CU FT	γ_{d_o}	101.2	*		
	SATURATION, %	S_o	42.1			
	VOID RATIO	e_o	0.695			
BEFORE SHEAR	WATER CONTENT, %	w_c	19.4			
	DRY DENSITY LB/CU FT	γ_{d_c}	111.8	*		
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.535			
	FINAL BACK PRESSURE, T/SQ FT	u_o	5.00			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.20			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.18			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	8.98			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	----			
INITIAL DIAMETER, IN.		D_o	2.86			
INITIAL HEIGHT, IN.		H_o	6.30			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 26 | PL 14 | PI 12 | Gs *2.75 | TYPE OF SPECIMEN IN-SITU | TYPE OF TEST SAT. Q

REMARKS: GS-VALUE-ASSUMED

* Variation in "initial" to "before shear" densities caused by consolidation under pressure saturation for requested "saturated Q-test".

PROJECT UPPER GUADALUPE RIVER

BORING NO. RC-93-1

SAMPLE NO. 121393

DEPTH/ELEV 0.0 - 2.5 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
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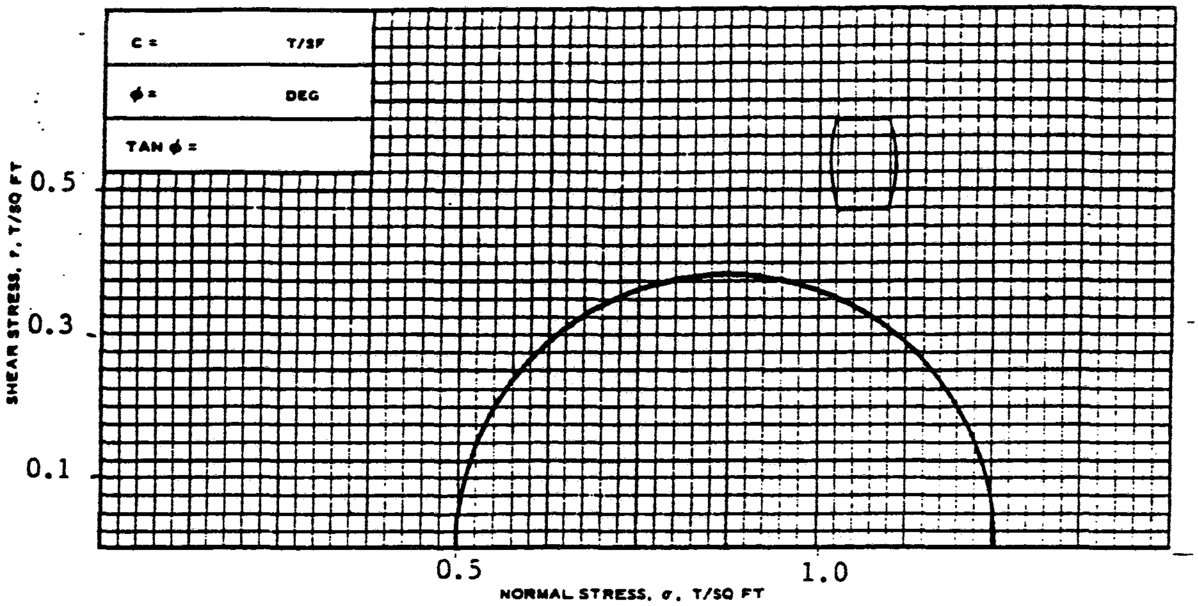
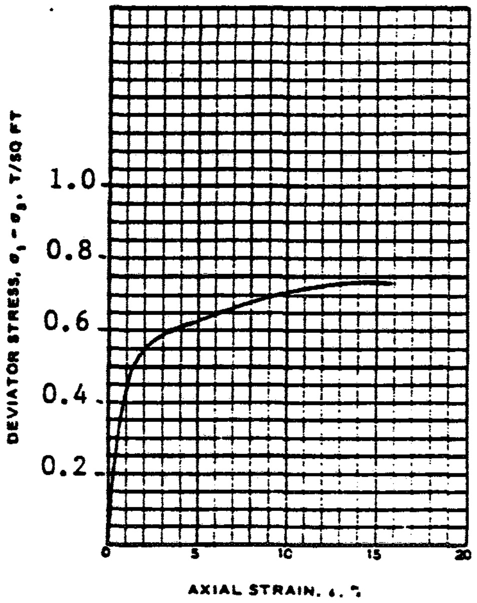
c = T/SF

φ = DEG

TAN φ =





SPECIMEN NO.			17			
INITIAL	WATER CONTENT, %	w_o	21.3			
	DRY DENSITY LB/CU FT	γ_{d_o}	102.2			
	SATURATION, %	S_o	86.2			
	VOID RATIO	e_o	0.679			
BEFORE SHEAR	WATER CONTENT, %	w_c	25.3			
	DRY DENSITY LB/CU FT	γ_{d_c}	101.3			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.695			
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.50			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.50			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.74			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	13.40			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	---			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.76			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 38 | PL 17 | PI 21 | Gs *2.75 | TYPE OF SPECIMEN IN-SITU | TYPE OF TEST SAT. Q

REMARKS: GS-VALUE-ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. RC-93-1

SAMPLE NO. 121394

DEPTH/ELEV 6.0 - 9.0 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

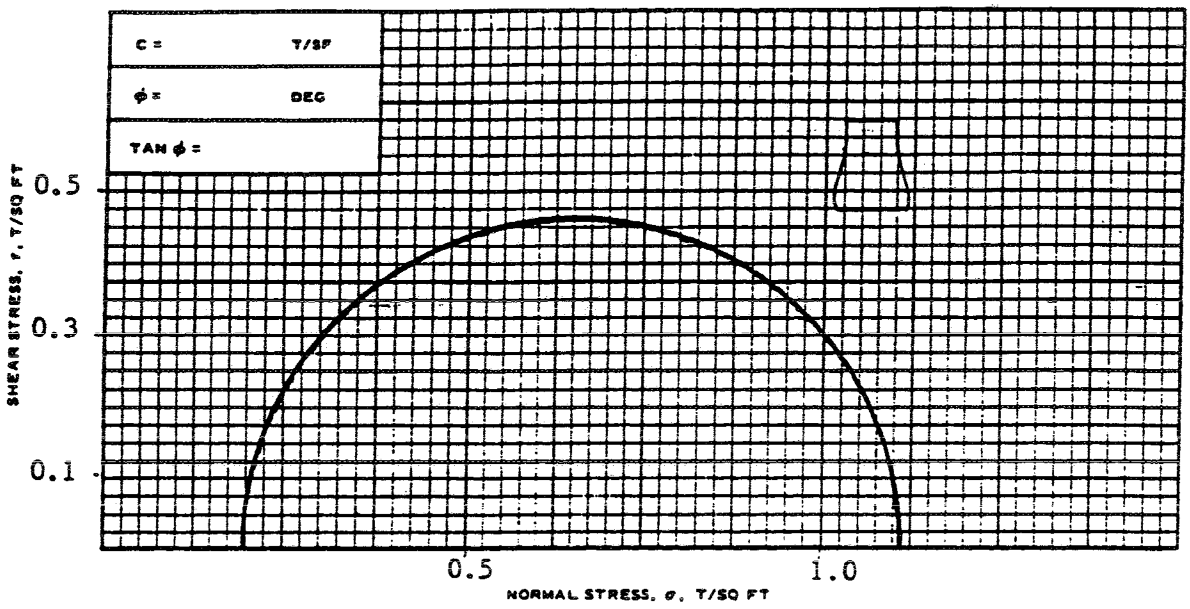
ENG FORM NO. 2089
REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE

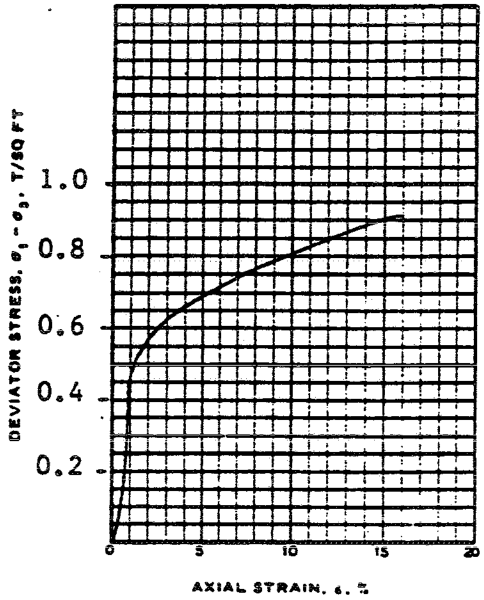
C37

C = T/SF

φ = DEG

TAN φ =





SPECIMEN NO.		18			
INITIAL	WATER CONTENT, % w_o	19.8			
	DRY DENSITY LB./CU FT γd_o	102.5			
	SATURATION, % S_o	80.9			
	VOID RATIO e_o	0.674			
BEFORE SHEAR	WATER CONTENT, % w_c	25.0			
	DRY DENSITY LB. CU FT γd_c	101.7			
	SATURATION, % S_c	100.0			
	VOID RATIO e_c	0.688			
	FINAL BACK PRESSURE, T/SQ FT u_o	5.90			
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	0.20			
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$	0.90			
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN	t_f	15.32			
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$	----			
INITIAL DIAMETER, IN.	D_o	2.80			
INITIAL HEIGHT, IN.	H_o	6.69			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 39 PL 16 PI 23 Gs *2.75

TYPE OF SPECIMEN IN-SITU

TYPE OF TEST SAT. Q

REMARKS: GS-VALUE-ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. RC-93-2

SAMPLE NO. 121396

DEPTH/ELEV 1.0 - 3.0 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
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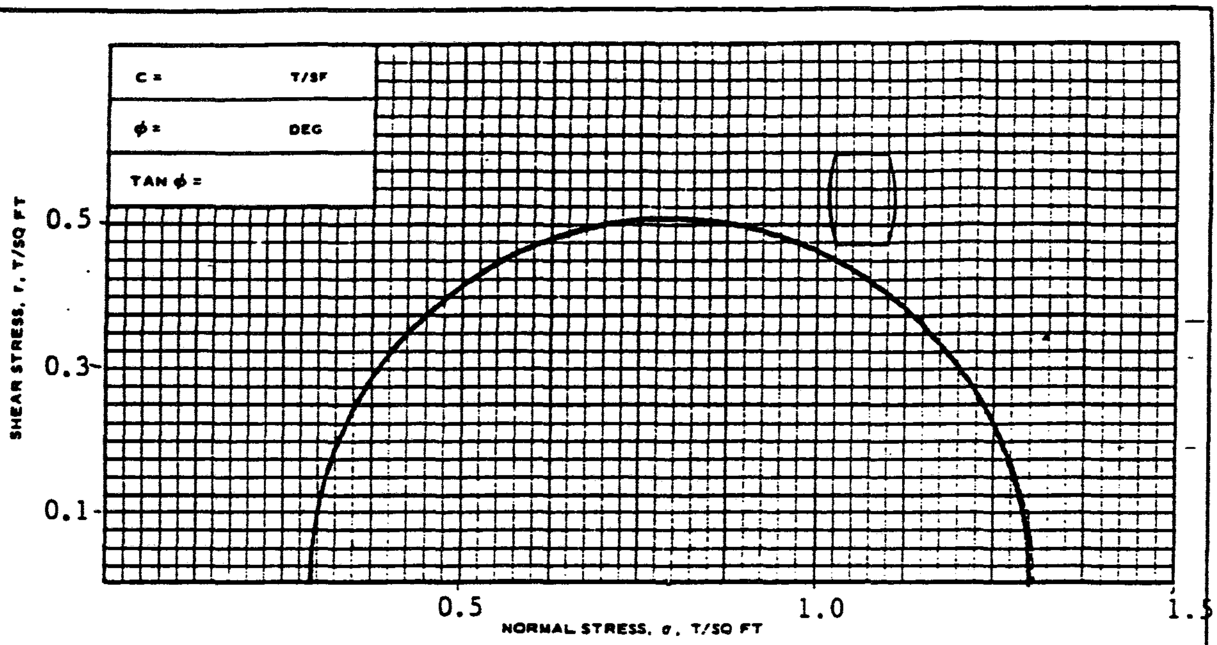
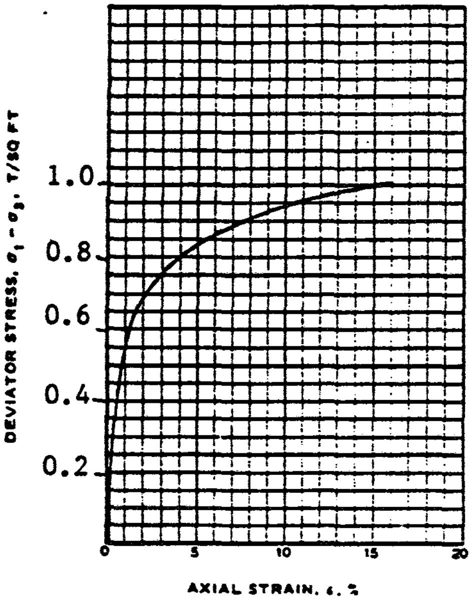
C38

C = T/SF

φ = DEG

TAN φ =





SPECIMEN NO.			19			
INITIAL	WATER CONTENT, %	w_o	14.6			
	DRY DENSITY LB/CU FT	γd_o	106.5			
	SATURATION, %	S_o	65.9			
	VOID RATIO	e_o	0.611			
BEFORE SHEAR	WATER CONTENT, %	w_c	22.8			
	DRY DENSITY LB/CU FT	γd_c	105.5			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.627			
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.50			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.29			
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$		1.00			
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	15.62			
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$		-----			
INITIAL DIAMETER, IN.		D_o	2.80			
INITIAL HEIGHT, IN.		H_o	6.35			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 37	PL 15	PI 22	Gs *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT Q

REMARKS: GS-VALUE-ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. RC-93-3

SAMPLE NO. 121397

DEPTH/ELEV 2.0 - 5.0 FT

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970

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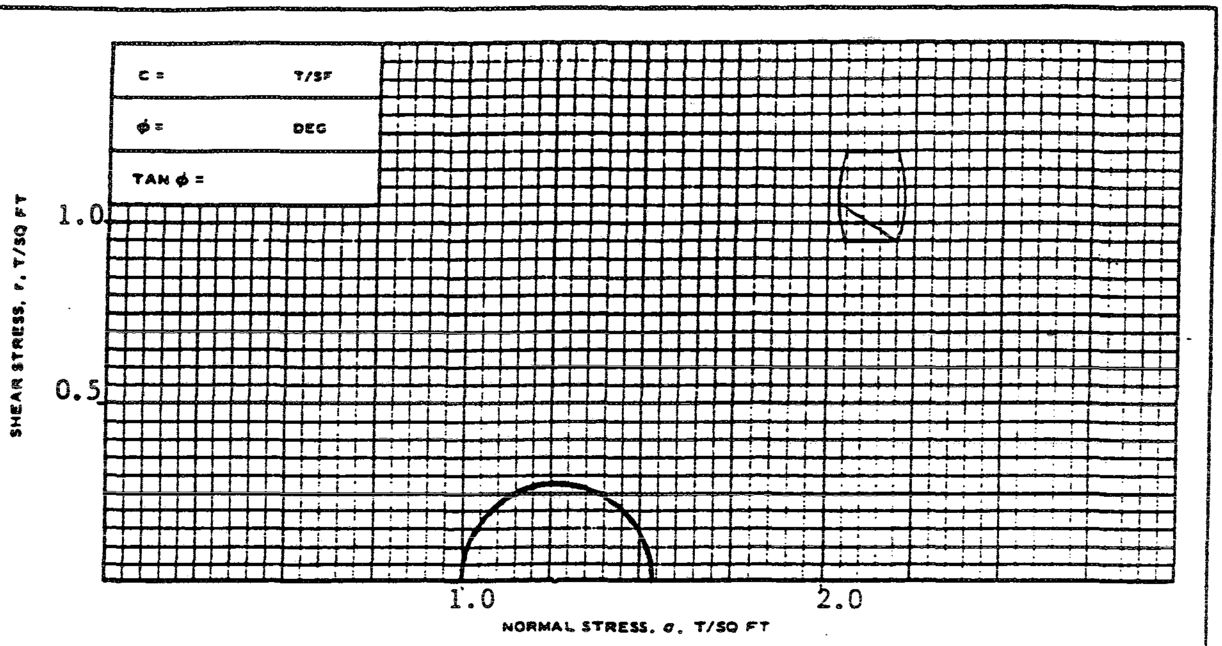
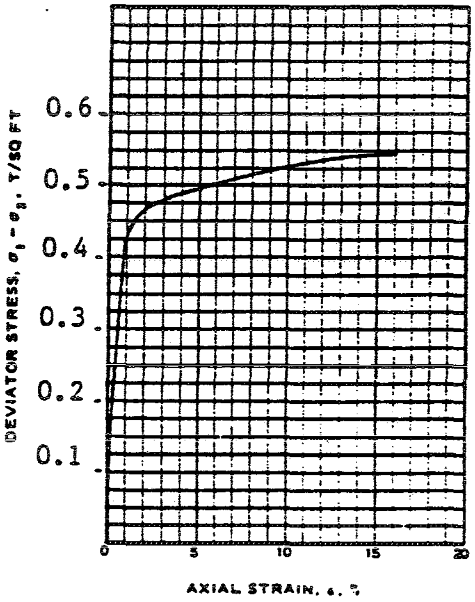


c = T/SF

φ = DEG

TAN φ =



SPECIMEN NO.			20			
INITIAL	WATER CONTENT, %	w_o	14.9			
	DRY DENSITY LB./CU FT	γd_o	98.8			
	SATURATION, %	S_o	55.5			
	VOID RATIO	e_o	0.736			
BEFORE SHEAR	WATER CONTENT, %	w_c	25.1			
	DRY DENSITY LB./CU FT	γd_c	101.5			
	SATURATION, %	S_c	100.0			
	VOID RATIO	e_c	0.691			
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.50			
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00			
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	0.54			
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	12.82			
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	---			
INITIAL DIAMETER, IN.		D_o	2.79			
INITIAL HEIGHT, IN.		H_o	6.73			

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS

LL 30	PL 15	PI 15	G: *2.75	TYPE OF SPECIMEN IN-SITU	TYPE OF TEST SAT. Q

REMARKS: GS-VALUE ASSUMED

PROJECT UPPER GUADALUPE RIVER

BORING NO. RC-93-3

SAMPLE NO. 121398

DEPTH/ELEV 16.0 - 18.0

LABORATORY SPD

DATE OCTOBER 1993

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
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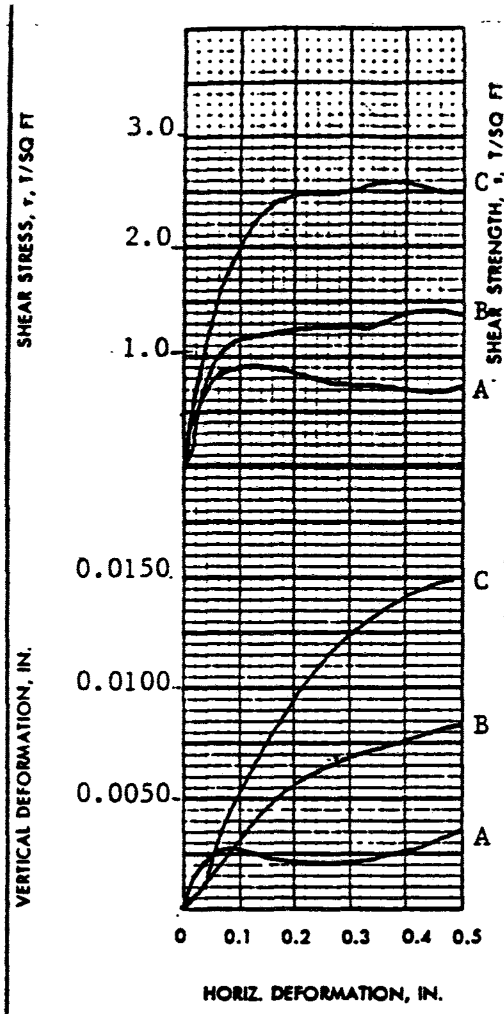
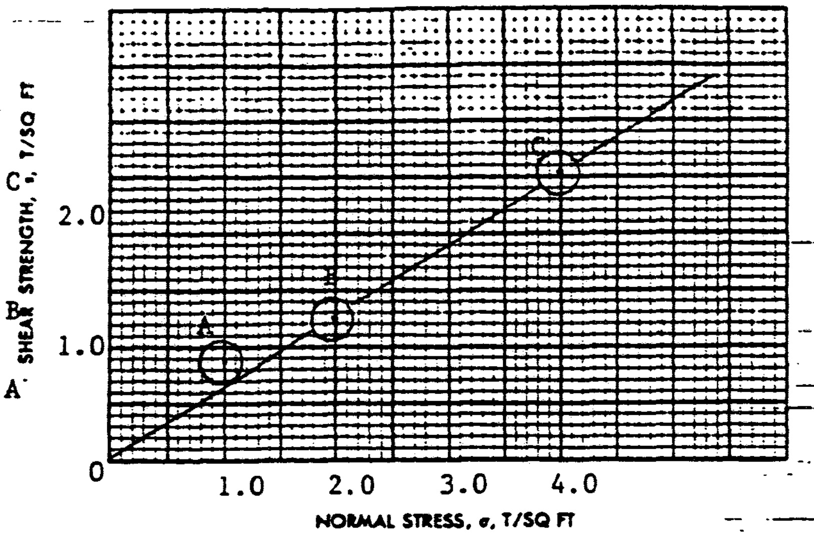




SHEAR STRENGTH PARAMETERS

φ' = ____________

TAN φ' = ____________

c' = ____________ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

				A	B	C	
TEST NO 121392				A	B	C	[illegible]
INITIAL	WATER CONTENT		w_o	19.9 %	20.1 %	19.7 %	19 = %
	VOID RATIO		e_o	0.579 0.551	0.630 0.601	[illegible] ~~0.582~~	0.60[illegible]
	SATURATION		S_o	94.5 ~~97~~ %	87.7 ~~98~~ %	85.6% 98 %	90.1 %
	DRY DENSITY, LB/CU FT		γ_d	108.7	105.3	106.5	[illegible]
VOID RATIO AFTER CONSOLIDATION			e_c	0.552 0.524	0.603 0.574	0.585 ~~0.556~~	0.5[illegible]
TIME FOR 50 PERCENT CONSOLIDATION, MIN			t_{50}	---	---	---	
FINAL	WATER CONTENT		w_f	19.2 %	21.0 %	20.3 %	%
	VOID RATIO		e_f	0.502 0.517	0.567	0.548	
	SATURATION		S_f	100 %	100 %	100 %	%
NORMAL STRESS, T/SQ FT			σ	1	2	4	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}		0.90	1.28	2.54	
ACTUAL TIME TO FAILURE, MIN			t_f	80	120	160	
RATE OF STRAIN, IN./MIN				0.0027	0.0027	0.0027	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}		0.81	1.27	2.48	

TYPE OF SPECIMEN undisturbed 2.5 IN.DIA. **IN. SQUARE** 0.5 **IN. THICK**

CLASSIFICATION (CL)

LL 30 **PL** 14 **PI** 16 **G$_s$** 2.7 (Est.)

REMARKS N/A

PROJECT UPPER GUADALUPE RIVER

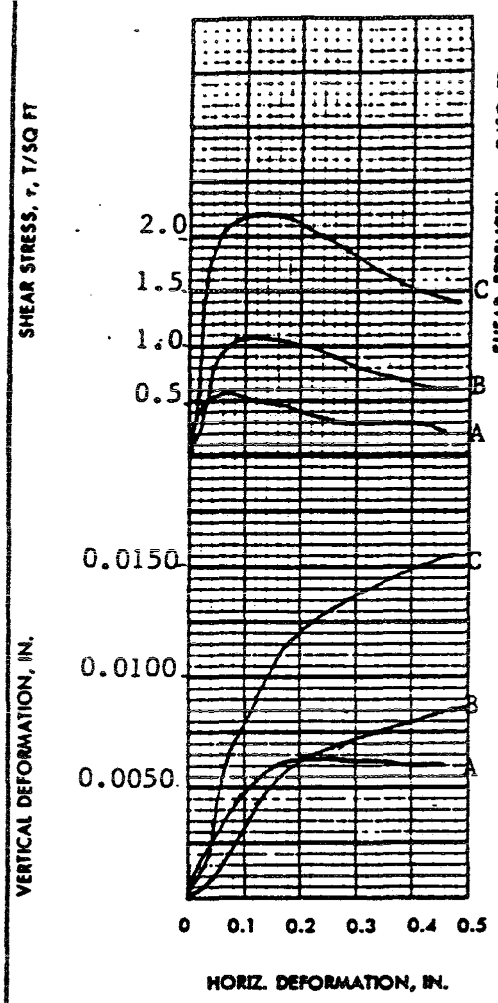
AREA

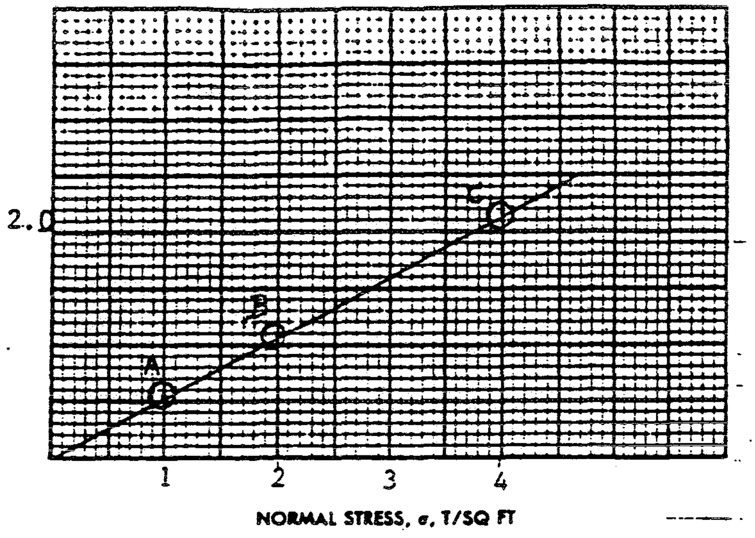
BORING NO. CC-93-2 **SAMPLE NO.** 8

DEPTH EL 25-28 **DATE** 10/93

DIRECT SHEAR TEST REPORT

c 41





SHEAR STRENGTH PARAMETERS

φ' = 27°

TAN φ' = 0.530

c' = 0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO. 121400			A	B	C	
INITIAL	WATER CONTENT	w_o	39.4 %	29.3 %	32.2 %	22.4 %
	VOID RATIO	e_o	1.130 ~~1.116~~	0.857 0.837	0.943 0.921	0.972
	SATURATION	S_o	94.5 ~~95~~ %	93.4 94 %	93 ~~94~~ %	93.8 %
	DRY DENSITY, LB/CU FT	γ_d	79.7	91.8	87.7	86.4
VOID RATIO AFTER CONSOLIDATION		e_c	1.080	0.806	0.889	0.937
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	50	50	50	
FINAL	WATER CONTENT	w_f	39.6%	29.5%	32.6%	%
	VOID RATIO	e_f	1.070	0.797	0.880	
	SATURATION	S_f	100 %	100 %	100 %	%
NORMAL STRESS, T/SQ FT		σ	1	2	4	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.54	1.08	2.18	
ACTUAL TIME TO FAILURE, MIN		t_f	60	60.	80	
RATE OF STRAIN, IN./MIN			0.0026	0.0033	0.0034	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	0.50	1.03	2.13	

TYPE OF SPECIMEN UNDISTURBED	2.5 IN.DIA.	IN. SQUARE	0.5	IN. THICK
CLASSIFICATION FAT CLAY				
LL 79	PL 26	PI 53		G_s 2.73 (EST.)

REMARKS N/A

PROJECT UPPER GUADALUPE RIVER

AREA

BORING NO. GR-93-3

SAMPLE NO. 8

DEPTH EL 20-23

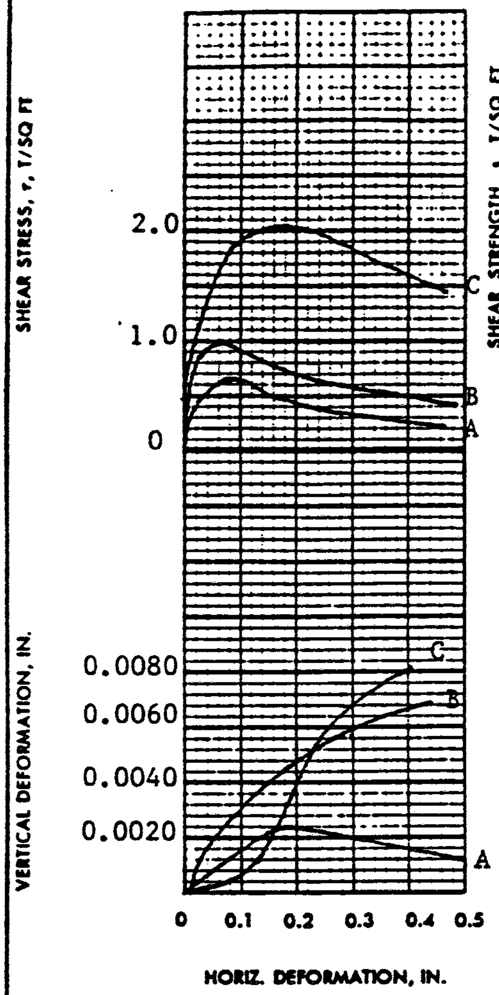
DATE 10/2/93

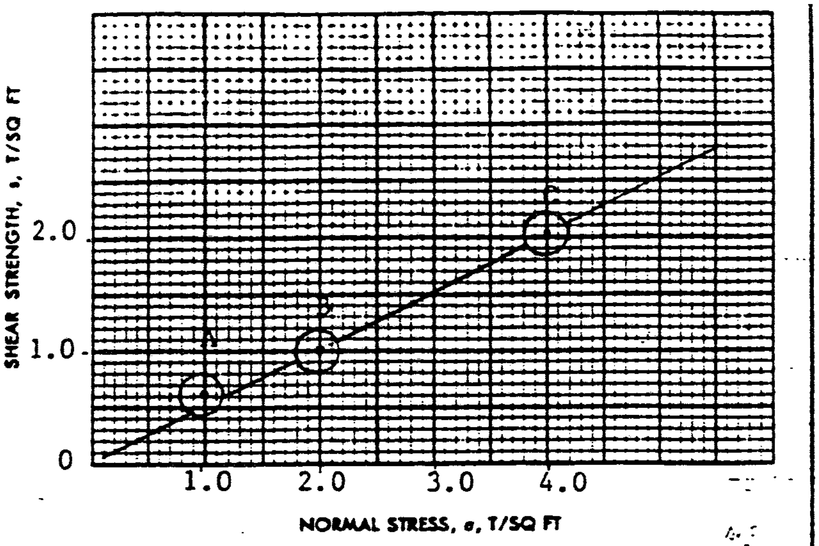
DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 **2092** (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)

GPO : 1980 O—210-040

PLATE IX-3





SHEAR STRENGTH PARAMETERS

φ' = ____________

TAN φ' = ____________

c' = ____________ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO. 121403		A	B	C	—
INITIAL — WATER CONTENT	w_o	33.7 %	32.6 %	33.1 %	33.2 %
INITIAL — VOID RATIO	e_o	0.928	0.871	0.954	0.946
INITIAL — SATURATION	S_o	98 %	101 %	94 %	96.2 %
INITIAL — DRY DENSITY, LB/CU FT	γ_d	87.4	90.1	86.3	87.9
VOID RATIO AFTER CONSOLIDATION	e_c	0.887	0.831	0.912	
TIME FOR 50 PERCENT CONSOLIDATION, MIN	t_{50}	---	---	---	
FINAL — WATER CONTENT	w_f	32.8 %	30.8 %	33.8 %	%
FINAL — VOID RATIO	e_f	0.887	0.831	0.912	
FINAL — SATURATION	S_f	100 %	100 %	100 %	%
NORMAL STRESS, T/SQ FT	σ	1	2	4	
MAXIMUM SHEAR STRESS, T/SQ FT	τ_{max}	0.64	0.99	2.05	---
ACTUAL TIME TO FAILURE, MIN	t_f	60	60.	80	
RATE OF STRAIN, IN./MIN		0.0025	0.0024	0.0031	
ULTIMATE SHEAR STRESS, T/SQ FT	τ_{ult}	0.54	0.86	1.83	

TYPE OF SPECIMEN UNDISTURBED 2.5 IN.DIA.	IN. SQUARE	0.5	IN. THICK
CLASSIFICATION			
LL 53	PL 19	PI 34	G_s 2.74 (EST)

REMARKS N/A

PROJECT UPPER GUADALUPE RIVER

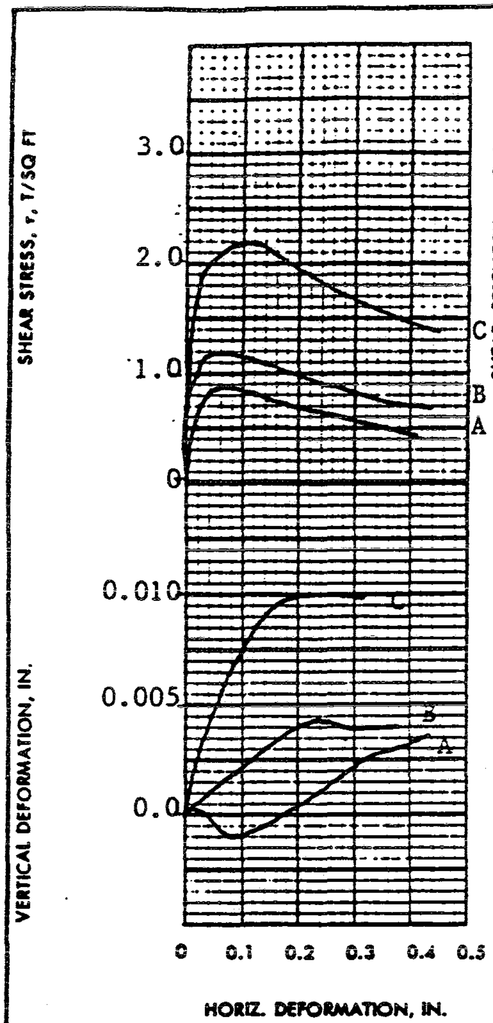
AREA

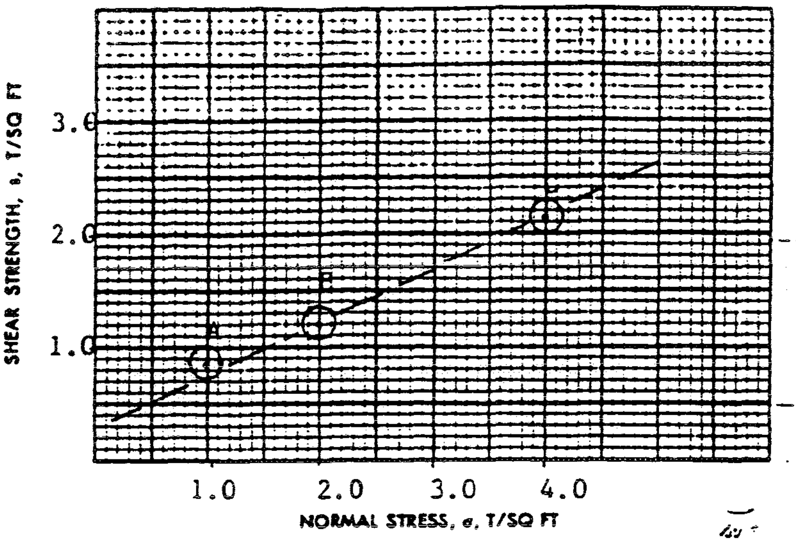
BORING NO. GR-93-2 | SAMPLE NO. 4

DEPTH EL 20 - 23 | DATE OCTOBER 1993

DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) PLATE IX-3





SHEAR STRENGTH PARAMETERS

ϕ' = [illegible] °

TAN ϕ' = [illegible]

c' = [illegible] T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO. 121406			A	B	C	
INITIAL	WATER CONTENT	w_o	30.5 %	30.3%	28.6 %	29.8 %
	VOID RATIO	e_o	0.870 0.837	0.875 0.837	0.826 0.794	0.856
	SATURATION	S_o	96.4 98 %	95.8 98 %	95.2 98 %	95.8 %
	DRY DENSITY, LB/CU FT	γ_d	91.8	91.8	94.0	92.5
VOID RATIO AFTER CONSOLIDATION		e_c	0.835 0.806	0.835 0.806	0.795 0.764	0.825
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	---	---	---	
FINAL	WATER CONTENT	w_f	29.5%	29.5%	28.0%	%
	VOID RATIO	e_f	0.797	0.797	0.756	
	SATURATION	S_f	100 %	100 %	100 %	%
NORMAL STRESS, T/SQ FT		σ	1	2	4	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.84	1.19	2.13	
ACTUAL TIME TO FAILURE, MIN		t_f	60	40	80	
RATE OF STRAIN, IN./MIN			0.0024	0.0024	0.0024	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	0.73	1.11	2.01	

TYPE OF SPECIMEN undisturbed	2.5 IN.DIA.	IN. SQUARE	0.5 IN. THICK
CLASSIFICATION Fat Clay			
LL 72	PL 24	PI 48	G_s 2.75 ~~2.70~~ (EST)

REMARKS N/A

PROJECT UPPER GUADALUPE RIVER

AREA

BORING NO. GR-93-4

SAMPLE NO. 5

DEPTH EL 20-23

DATE 10/93

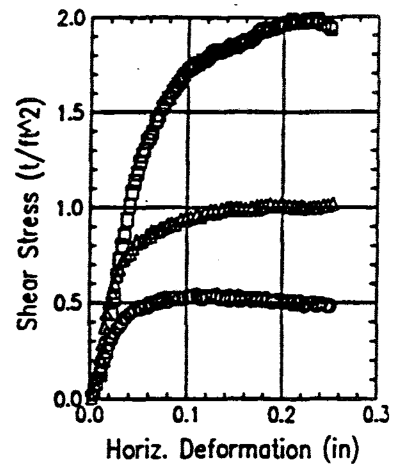
DIRECT SHEAR TEST REPORT

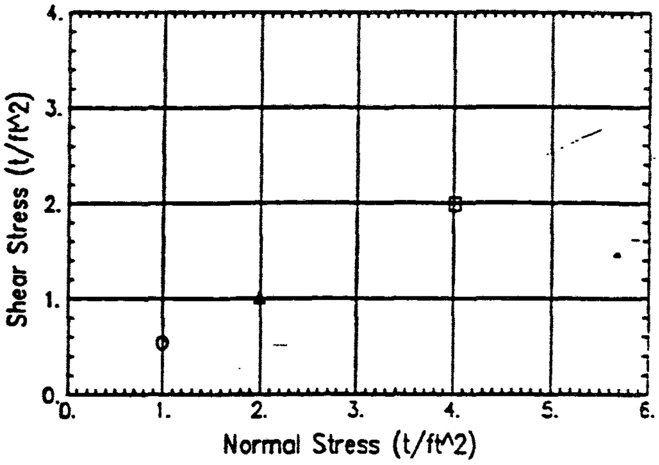
ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)

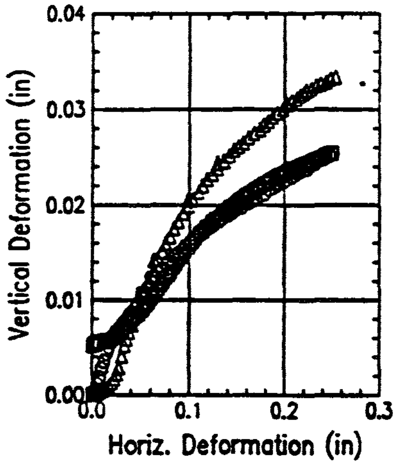
GPO : 1966 OF—216-045

PLATE IX-3

C44







Strength Parameters

Graph Symbol 121398			○	△	□	
Test No.			1	2	3	
Initial	Water content (%)	w_o	15.13	16.55	31.96	20.85
	Void ratio	e_o	0.90	0.95	1.13	1.017
	Saturation (%)	S_o	45.47	47.03	76.41	56.15
	Dry density (lb/ft³)	γ_d	88.78	86.44	79.16	84.79
Void ratio after consolidation		e_c	0.87	0.91	0.86	0.[illegible]
Time for 50 percent consolidation		t_{50}				
Final	Water content (%)	w_f	30.17	32.45	31.37	
	Void Ratio	e_f	0.82	0.85	0.80	
	Saturation (%)	S_f	98.84	103.10	105.34	
Normal stress (t/ft²)		σ	0.99	2.00	4.01	
Maximum shear stress (t/ft²)		τ_{max}	0.54	1.02	1.99	
Actual time to failure (min)		t_f	95	143	184	
Rate of strain						
Ultimate shear stress (t/ft²)		τ_{ult}	0.49	1.02	1.94	

Type of Specimen undisturbed	2.5 IN.DIA.		0.5 IN.THICK
Description			
LL 30	PL 15	PI 15		G$_s$ 2.7

Remarks	Project	Upper Gradualpue River	
	Area 4.91 (in²)	Boring No.	RC-93-3
	Depth 16-18	Sample No.	5
	Elevation	Date	10/26/93
	DIRECT SHEAR TEST REPORT		

C45

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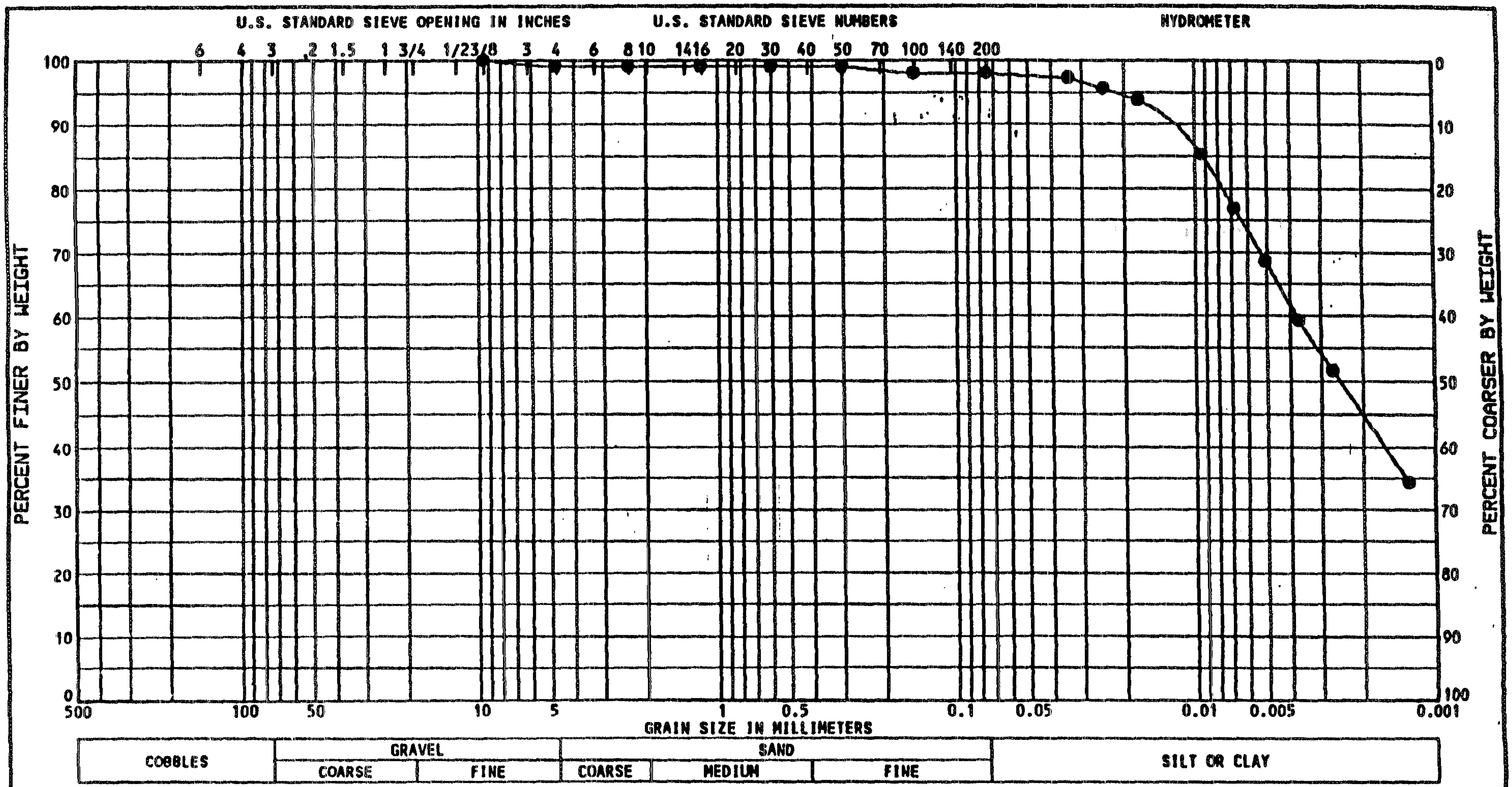

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121402 @ 10.0	FAT CLAY CH		62	21	41

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-2**

Notes **F.S. 3**

Date **10/22/93**

C46

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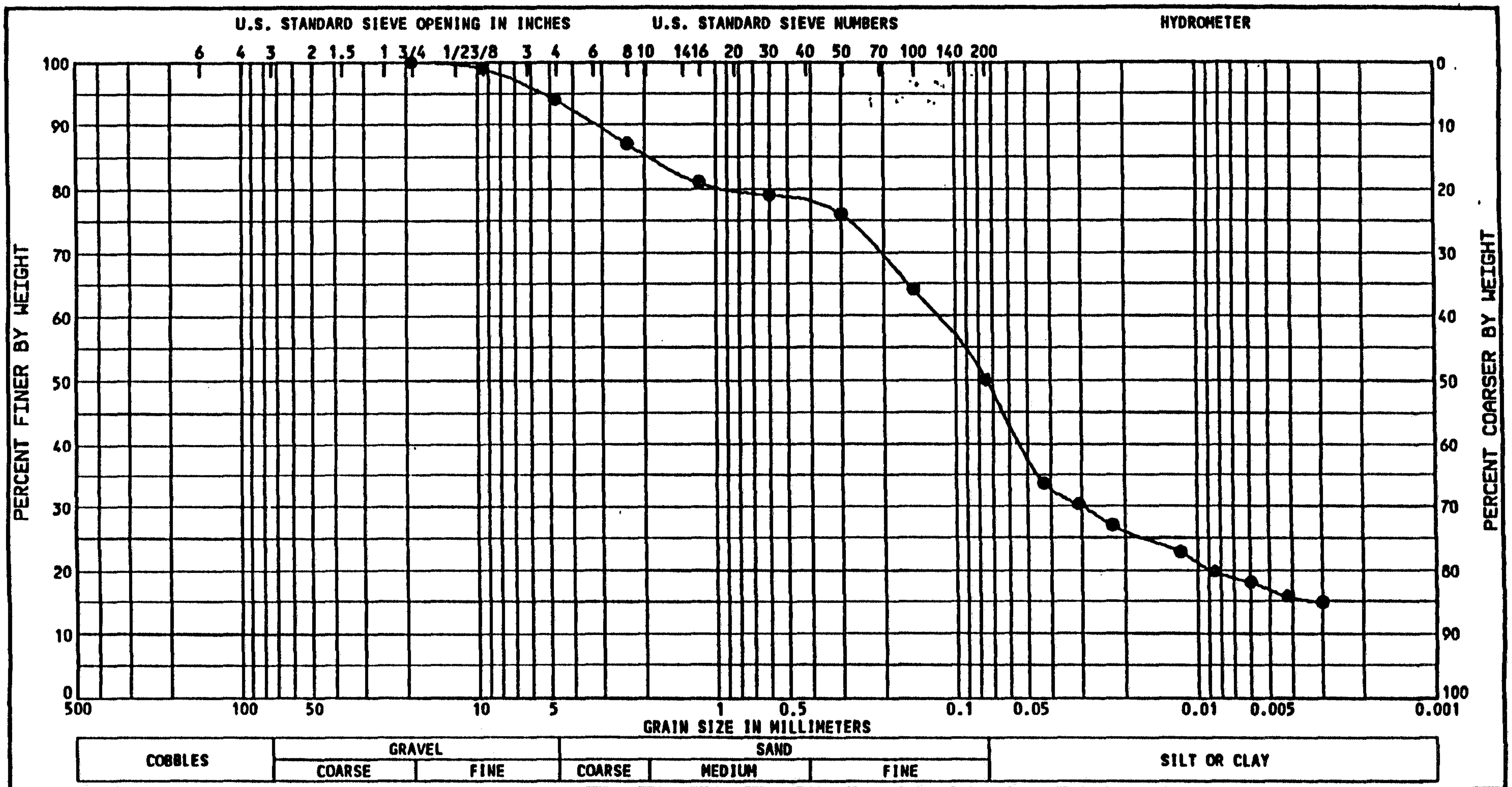
CORPS OF ENGINEERS

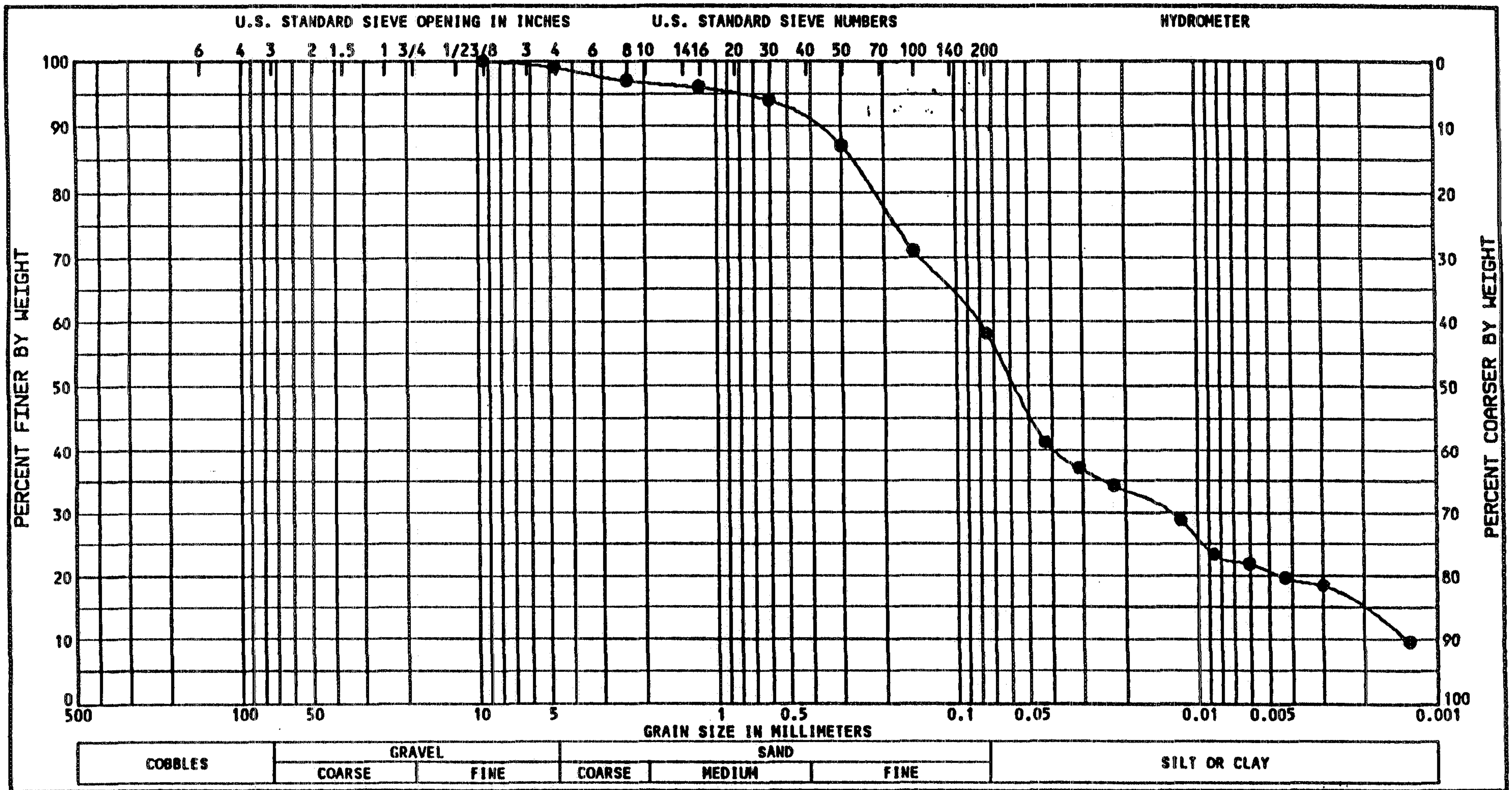


Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121300 @ 3.0					

GRADATION CURVES

Project Upper Guadalupe River
Santa Clara County
Area GR-93-3
Notes F.S. 1
Date 10/22/93

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121301 @ 4.5	SANDY LEAN CLAY CL		30	17	13

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-3**

Notes **F.S. 2**

Date **10/26/93**

C48

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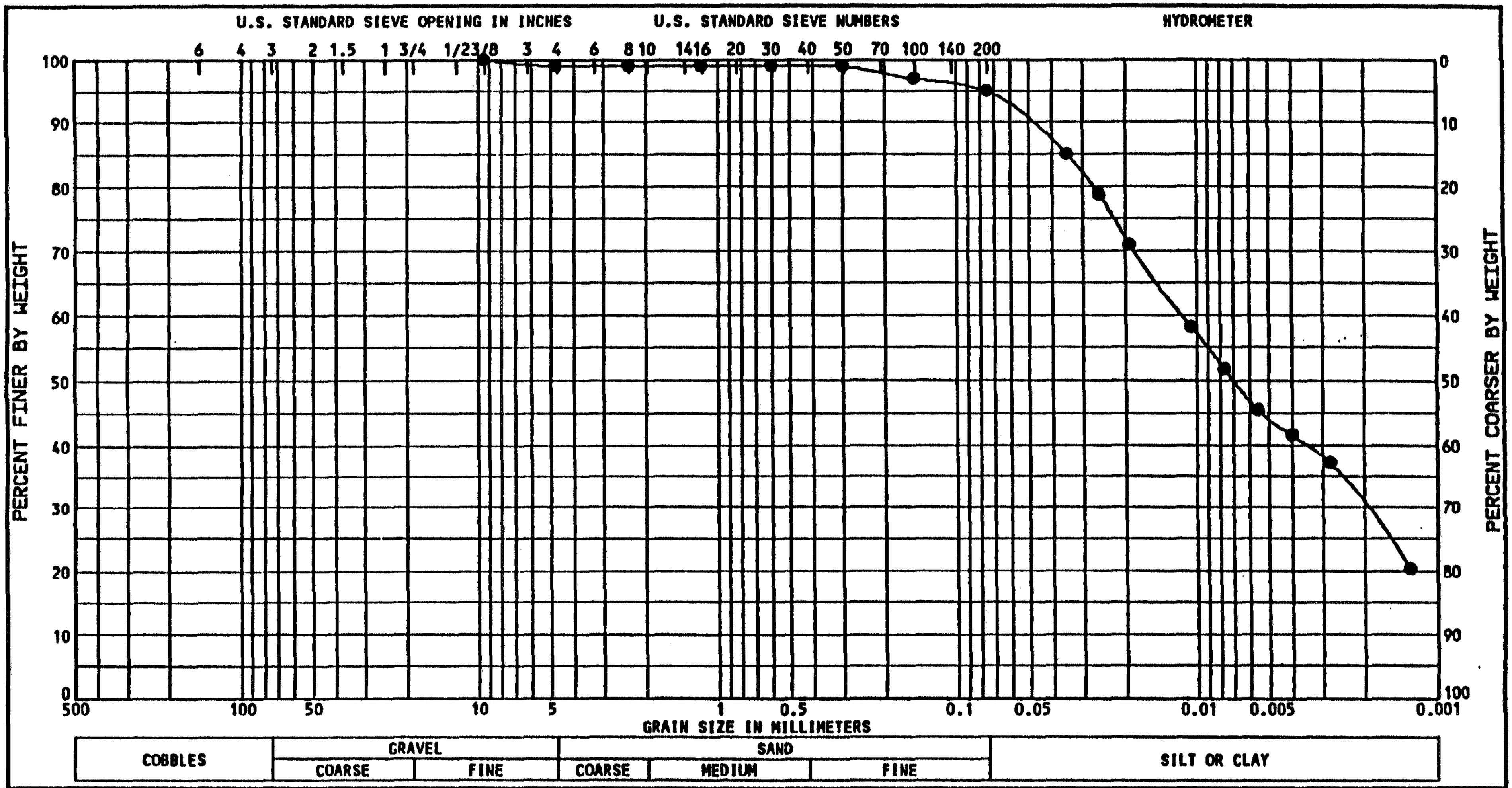

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121302 @ 7.5	FAT CLAY CH		52	18	34

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-3**

Notes **F.S. 6**

Date **10/27/93**

C49

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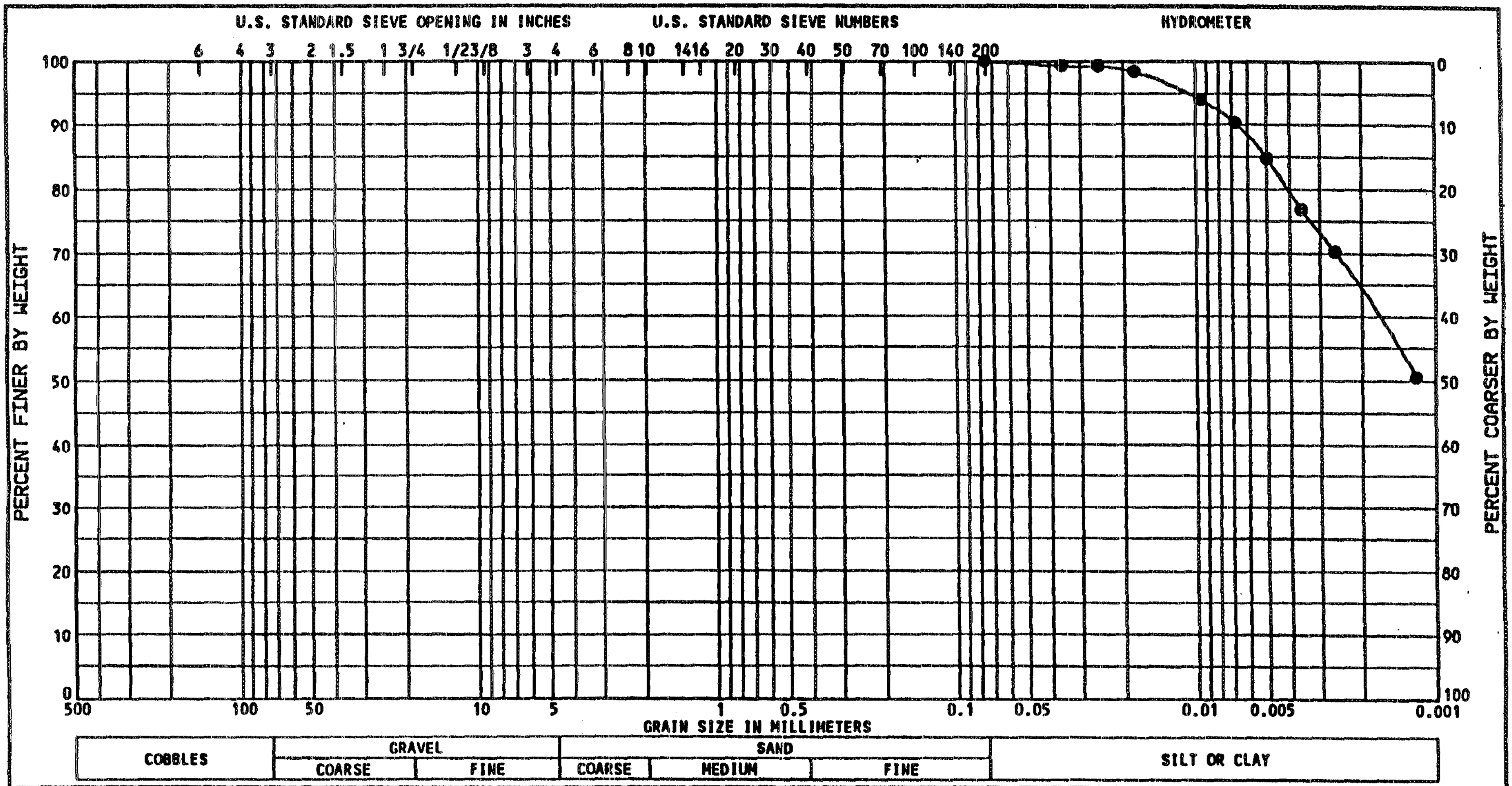

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121400 @ 20.0	FAT CLAY CH		79	26	53

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County
Area GR-93-3
Notes F.S. 23
Date 10/22/93

C50

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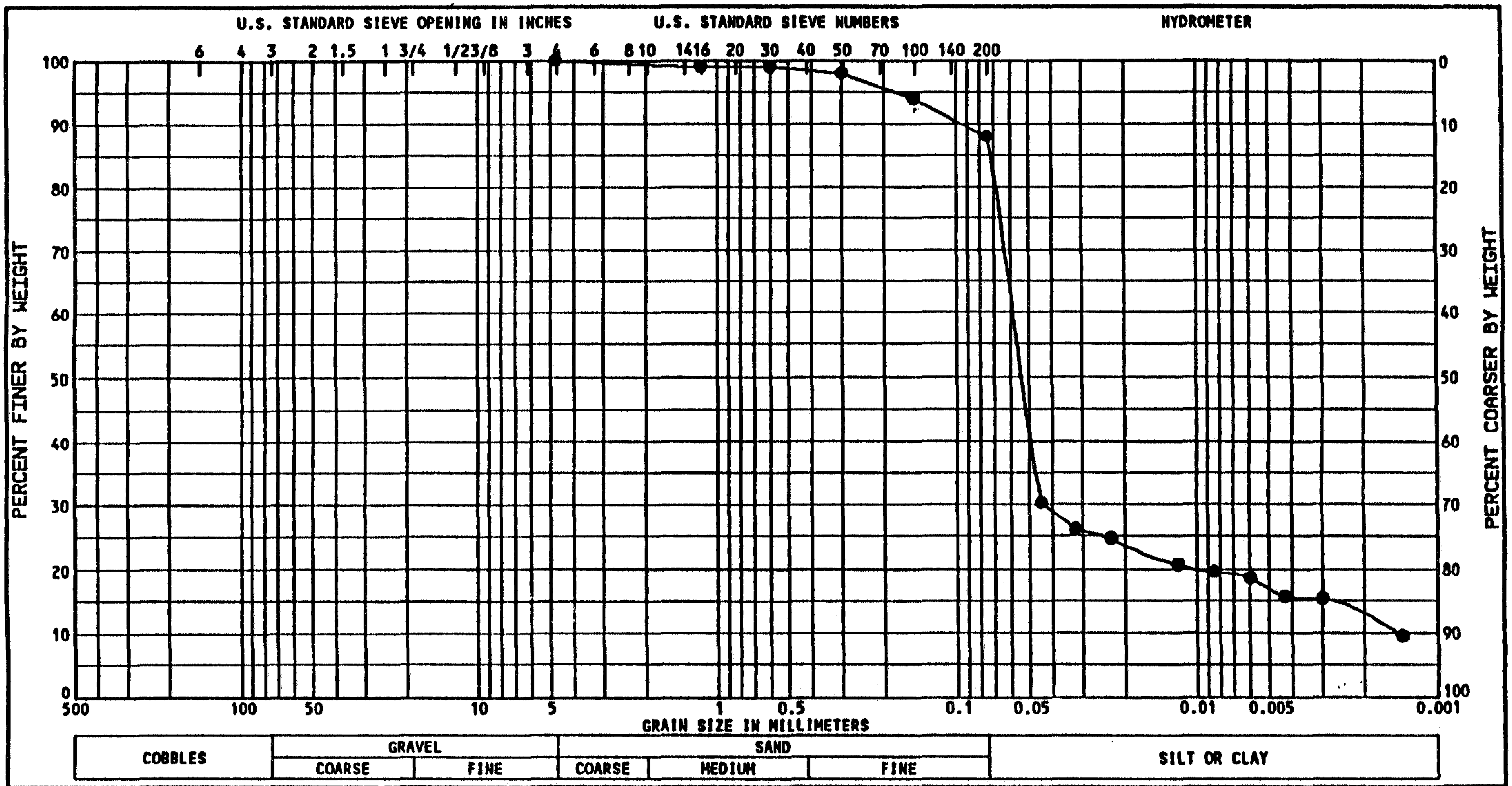

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121307 @ 12.0					

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-4**

Notes **F.S. 3**

Date **10/26/93**

GRADATION CURVES

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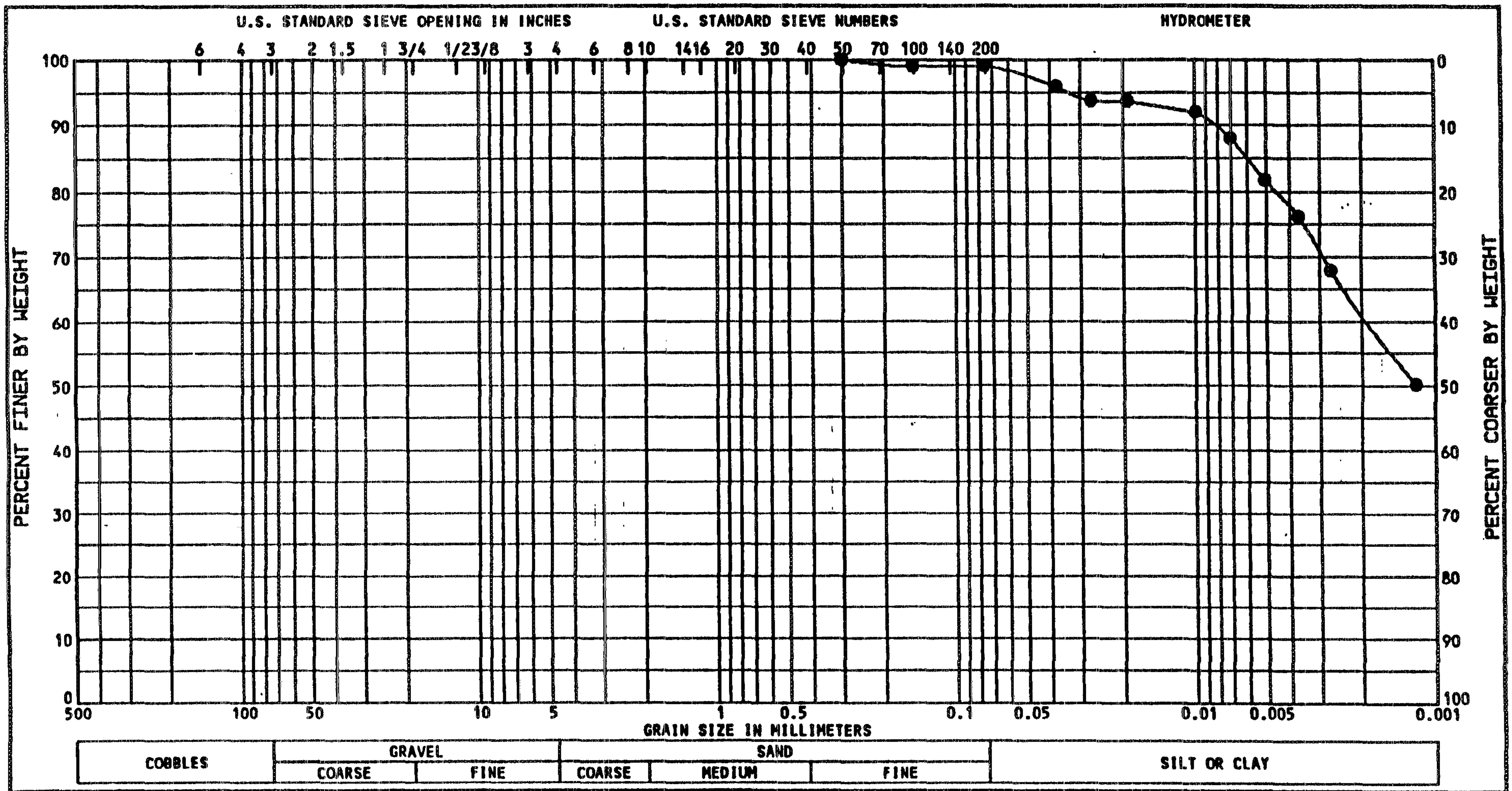

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121406 @ 20.0	FAT CLAY CH		72	24	48

Project Upper Guadalupe River
Santa Clara County

Area GR-93-4

Notes F.S. 5

Date 10/27/93

GRADATION CURVES

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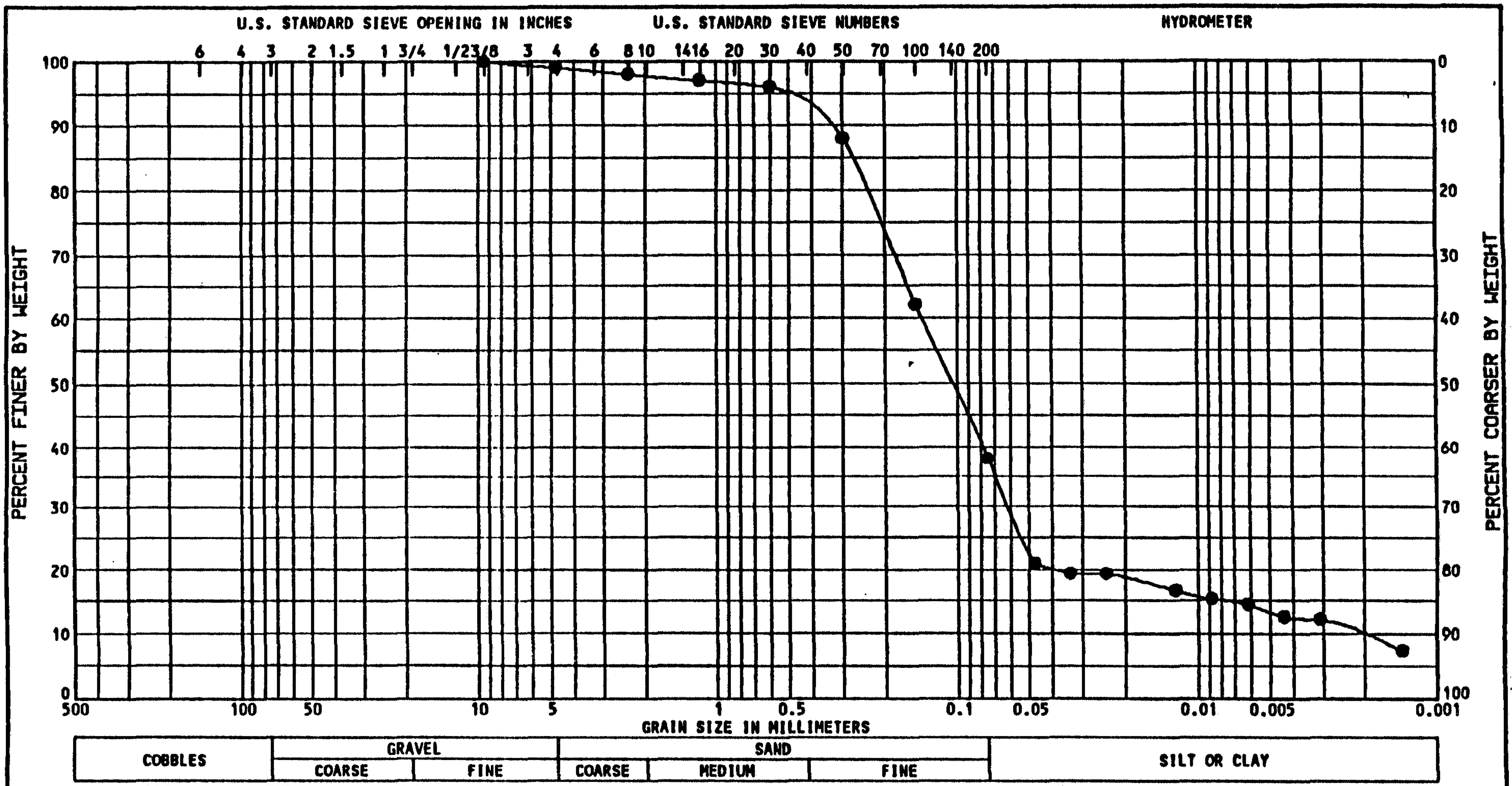

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121309 @ 5.5					

Project	**Upper Guadalupe River**
	Santa Clara County
Area	**GR-93-5**
Notes	**F.S. 2**
Date	**10/26/93**

GRADATION CURVES

C53

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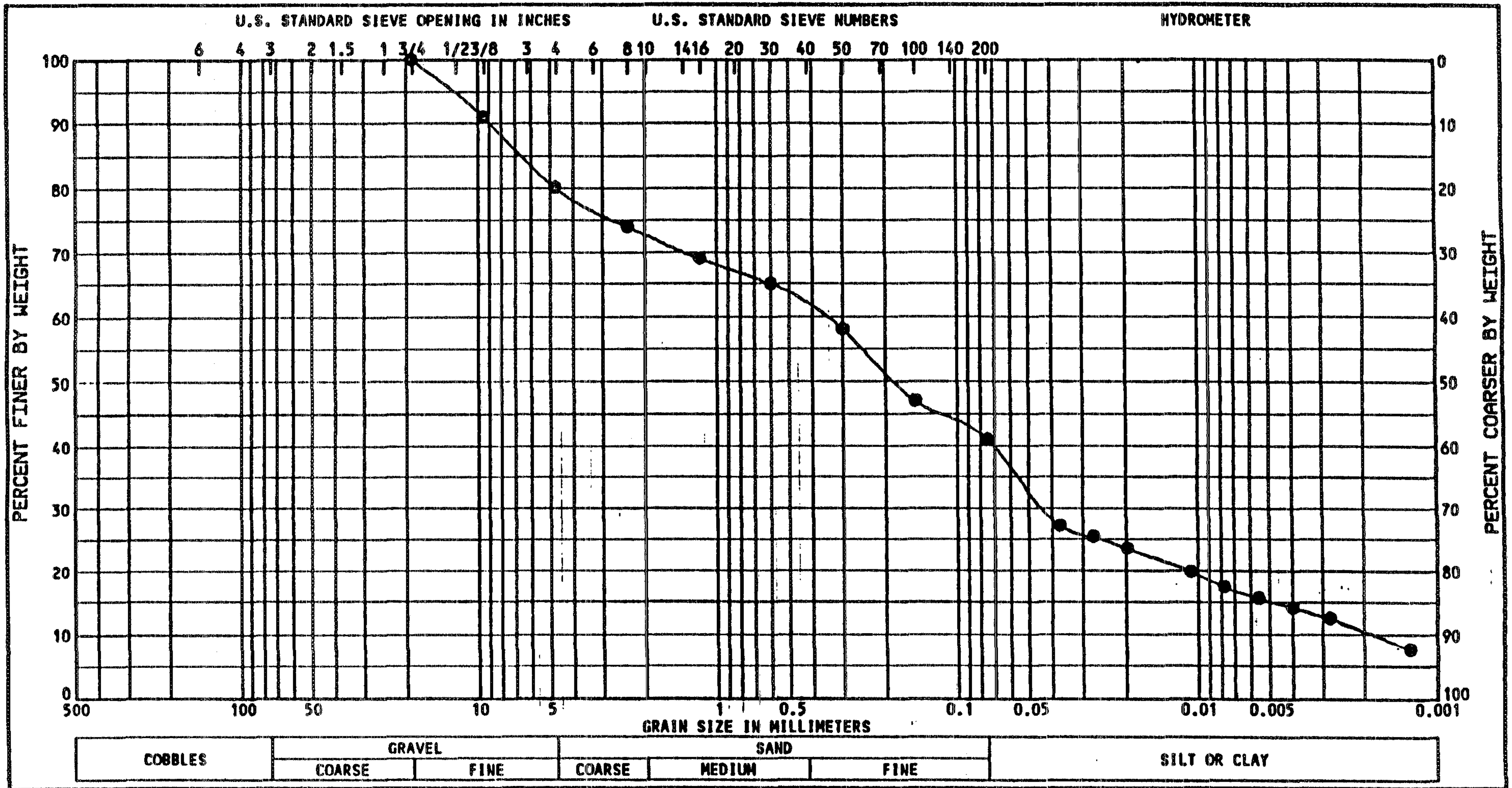

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121310 @ 7.0	CLAYEY SAND with GRAVEL SC		25	16	9

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-5**

Notes **F.S. 3**

Date **10/26/93**

C54

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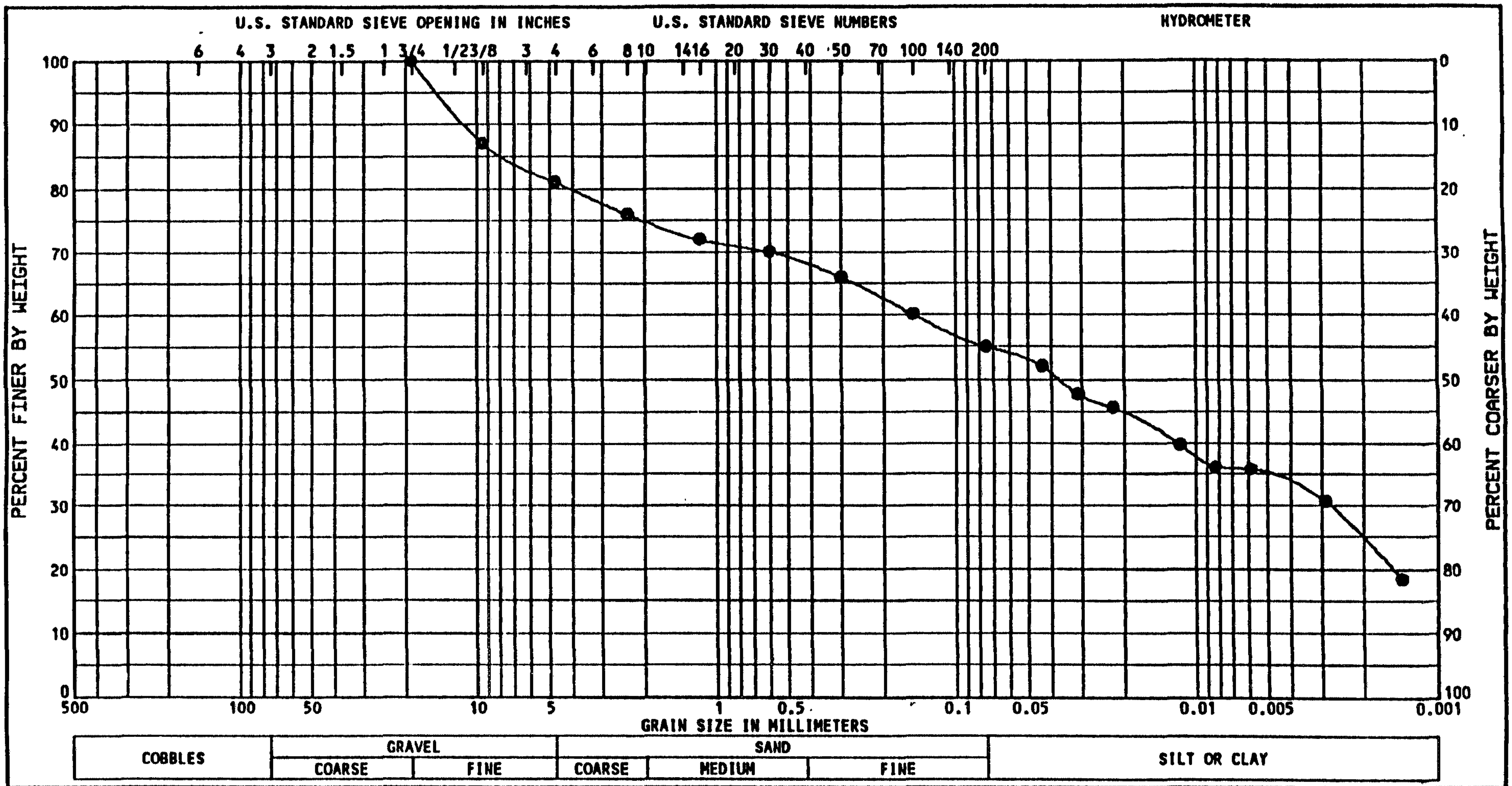

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121312 @ 18.5	SANDY LEAN CLAY with GRAVEL CL		28	14	14

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County
Area **GR-93-5**
Notes **F.S 6**
Date **10/25/93**

C55

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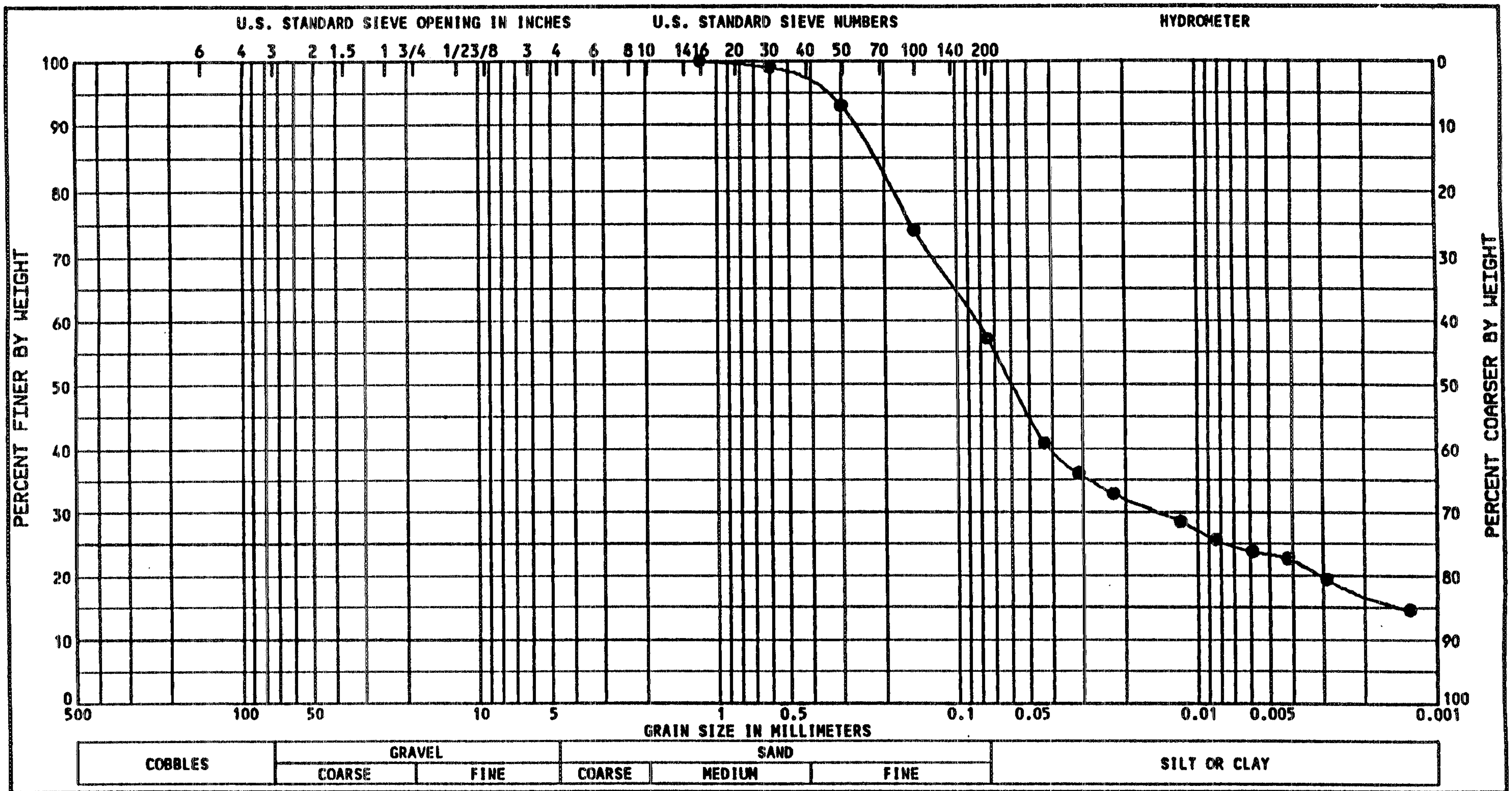

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121319 @ 5.5	SANDY LEAN CLAY CL		27	15	12

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-6**

Notes **F.S. 2**

Date **10/25/93**

GRADATION CURVES

C56

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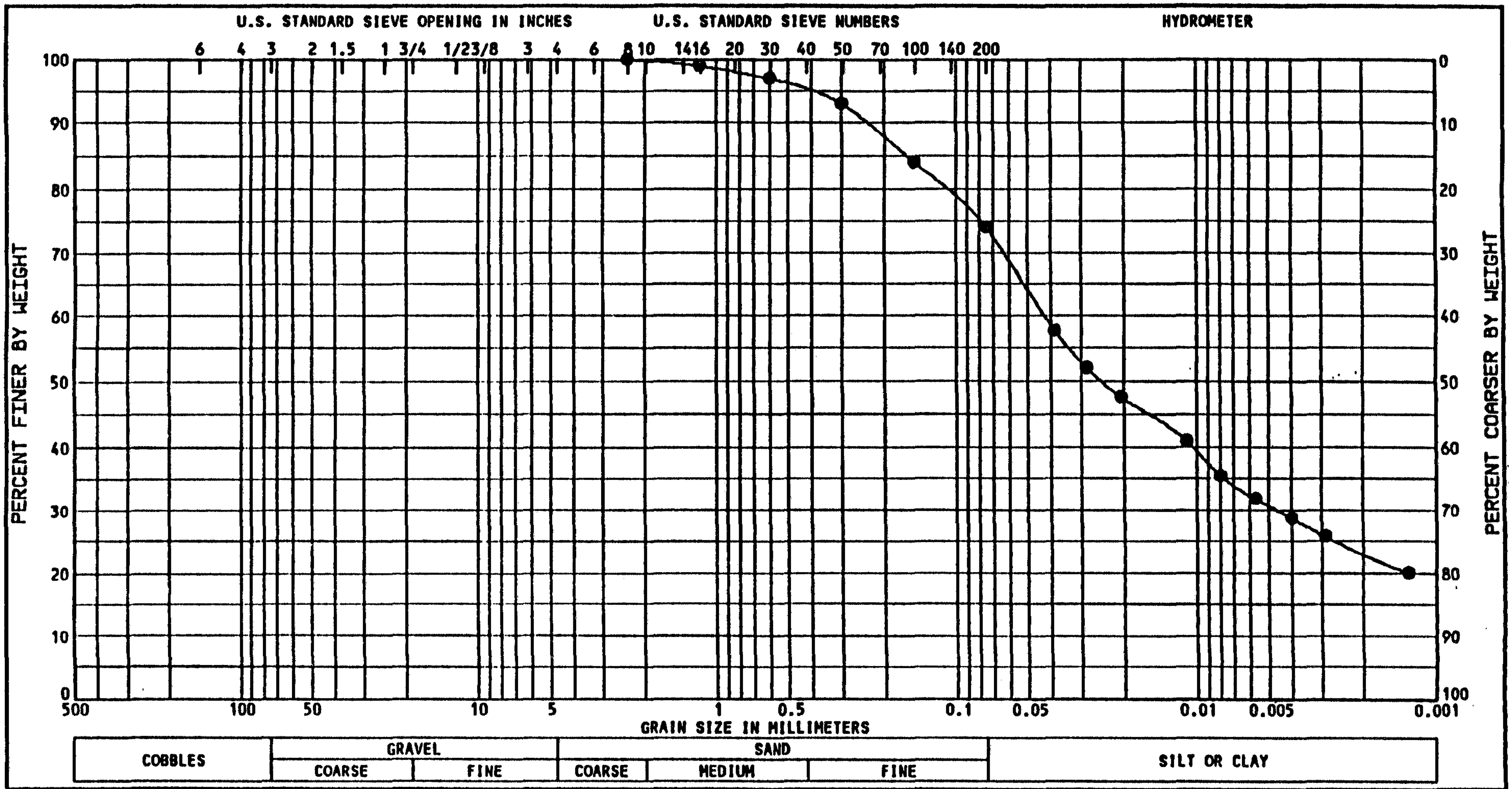

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121318 @ 7.0	LEAN CLAY with SAND CL		33	18	15

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-6**

Notes **F.S. 3**

Date **10/25/93**

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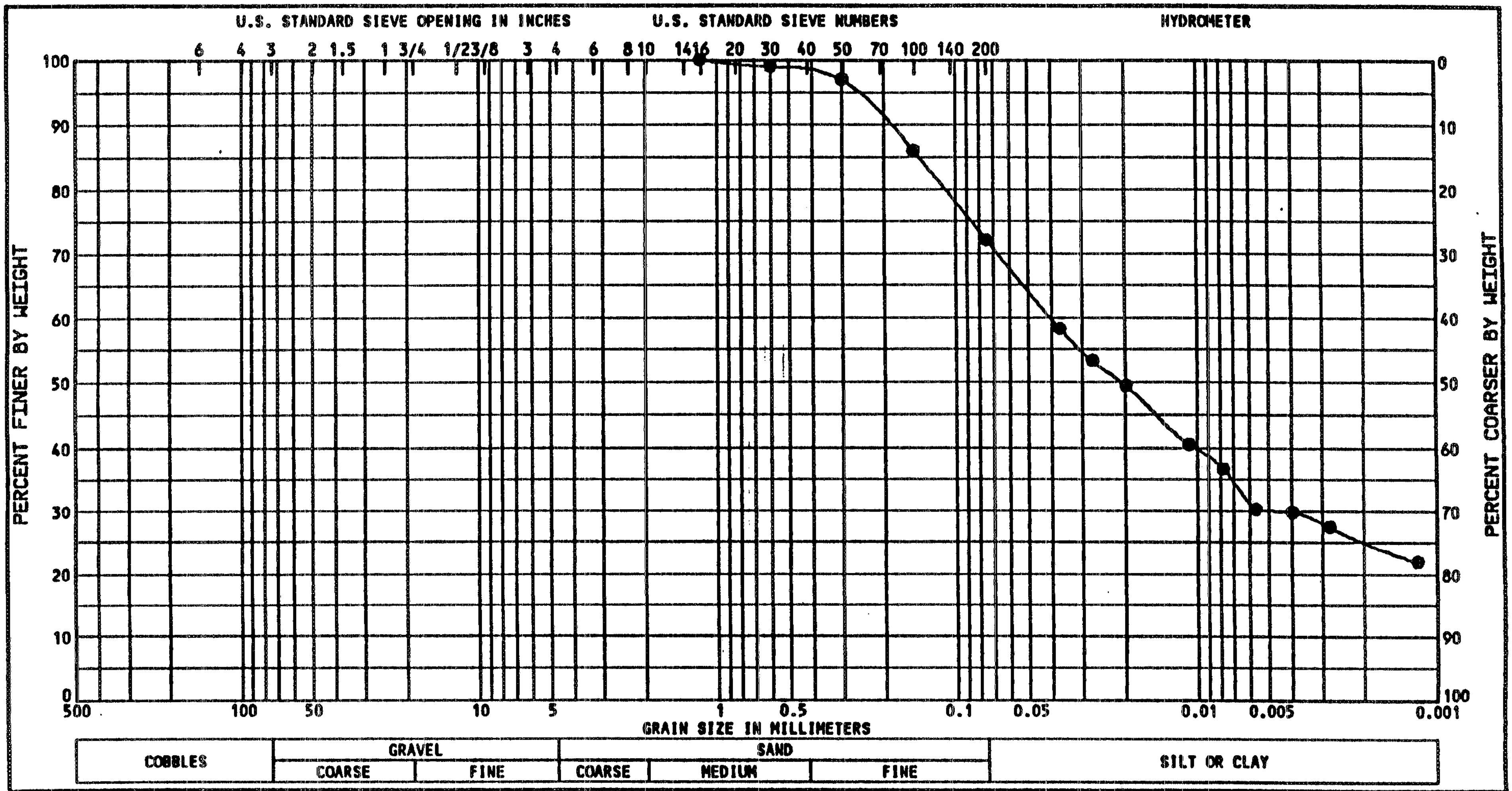

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121316 @ 14.0	LEAN CLAY with SAND CL		30	15	15

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-6**

Notes **F.S. 6**

Date **10/25/93**

C58

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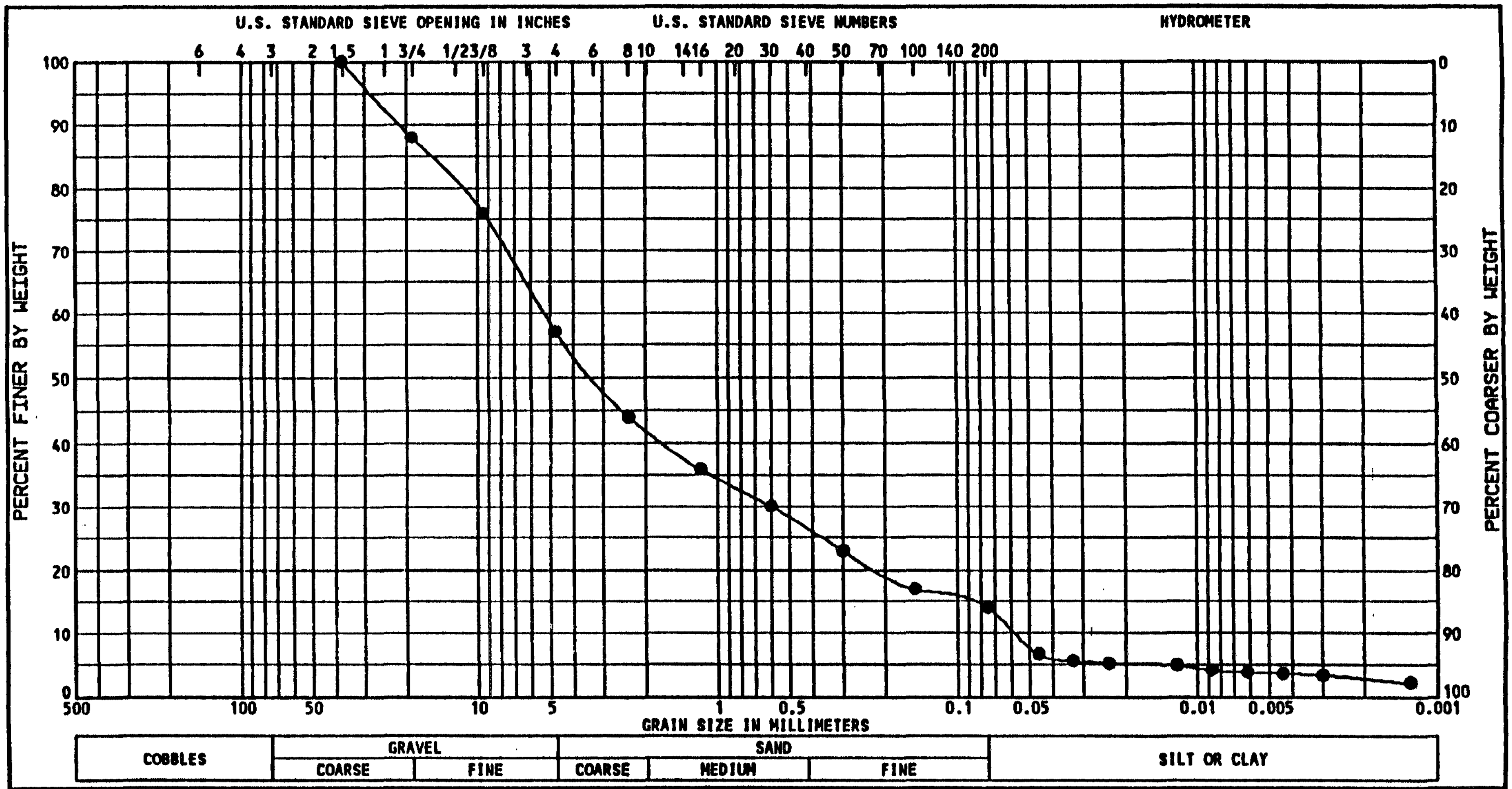

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121315 @ 19.5	CLAYEY GRAVEL with SAND GC		29	14	15

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-6**

Notes **F.S. 8**

Date **10/25/93**

C59

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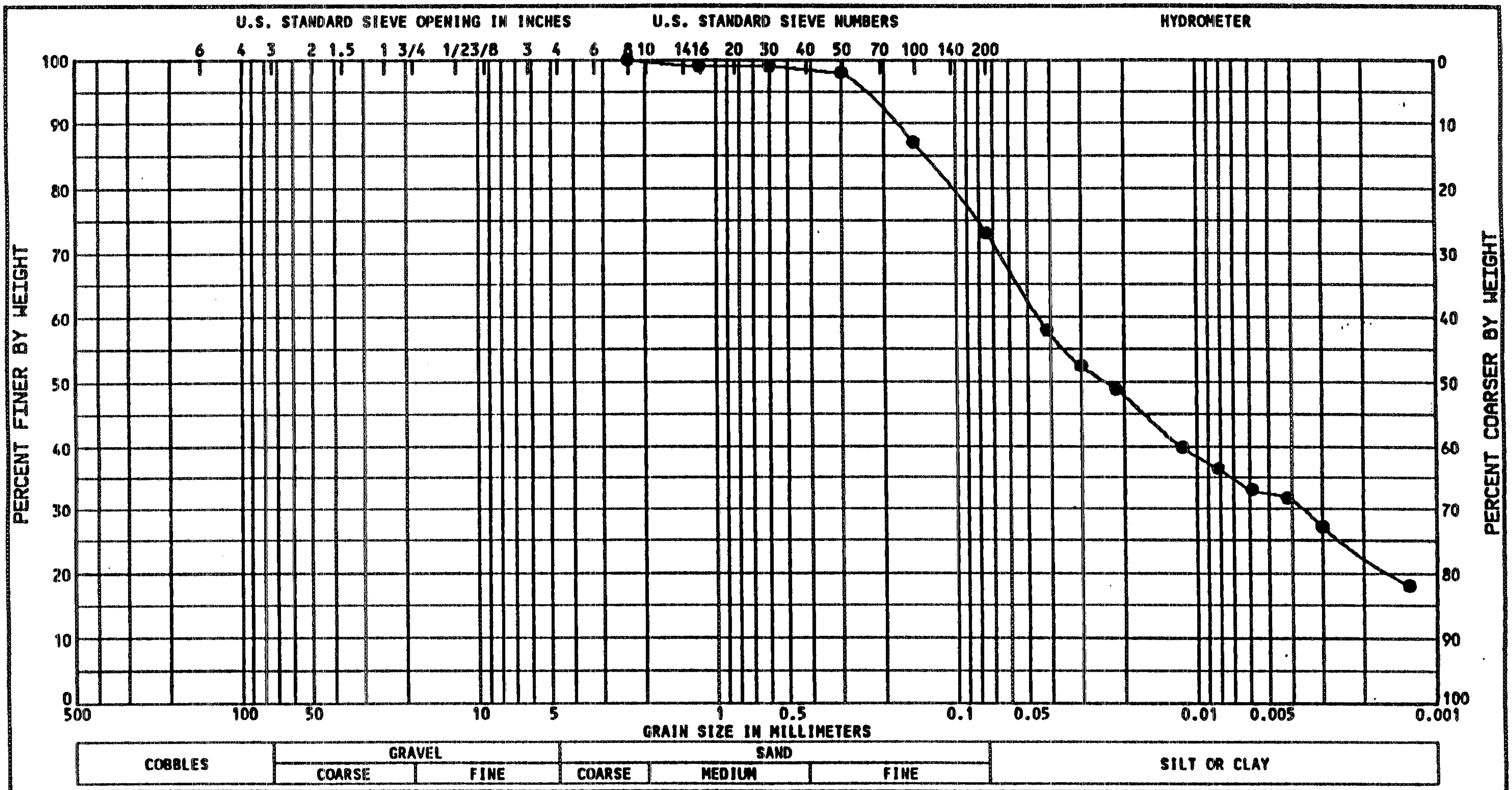


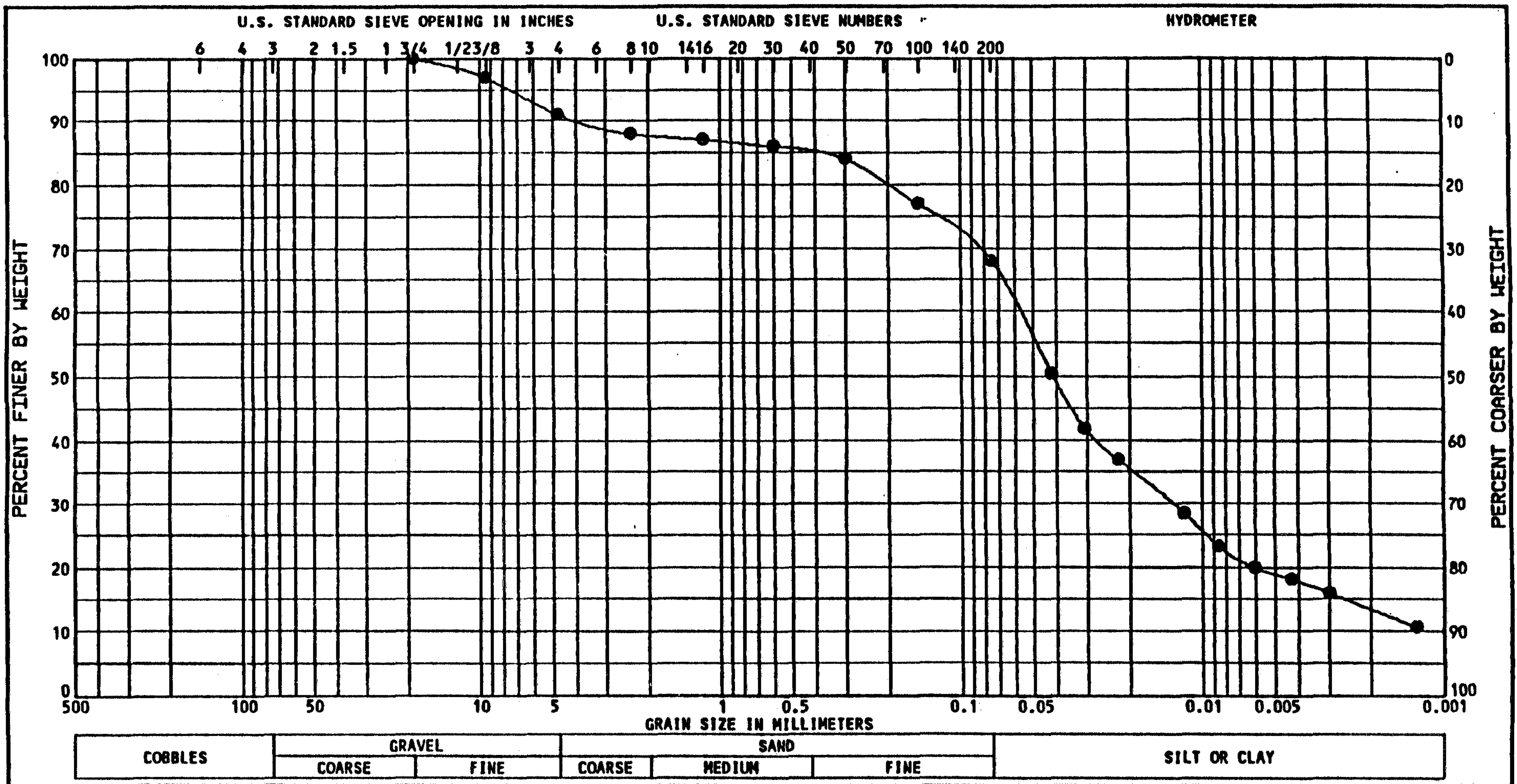
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121408 @ 25.0	LEAN CLAY with SAND CL		31	15	16

GRADATION CURVES

Project Upper Guadalupe River
Santa Clara County
Area GR-93-6
Notes F.S. 9
Date 10/27/93

097

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

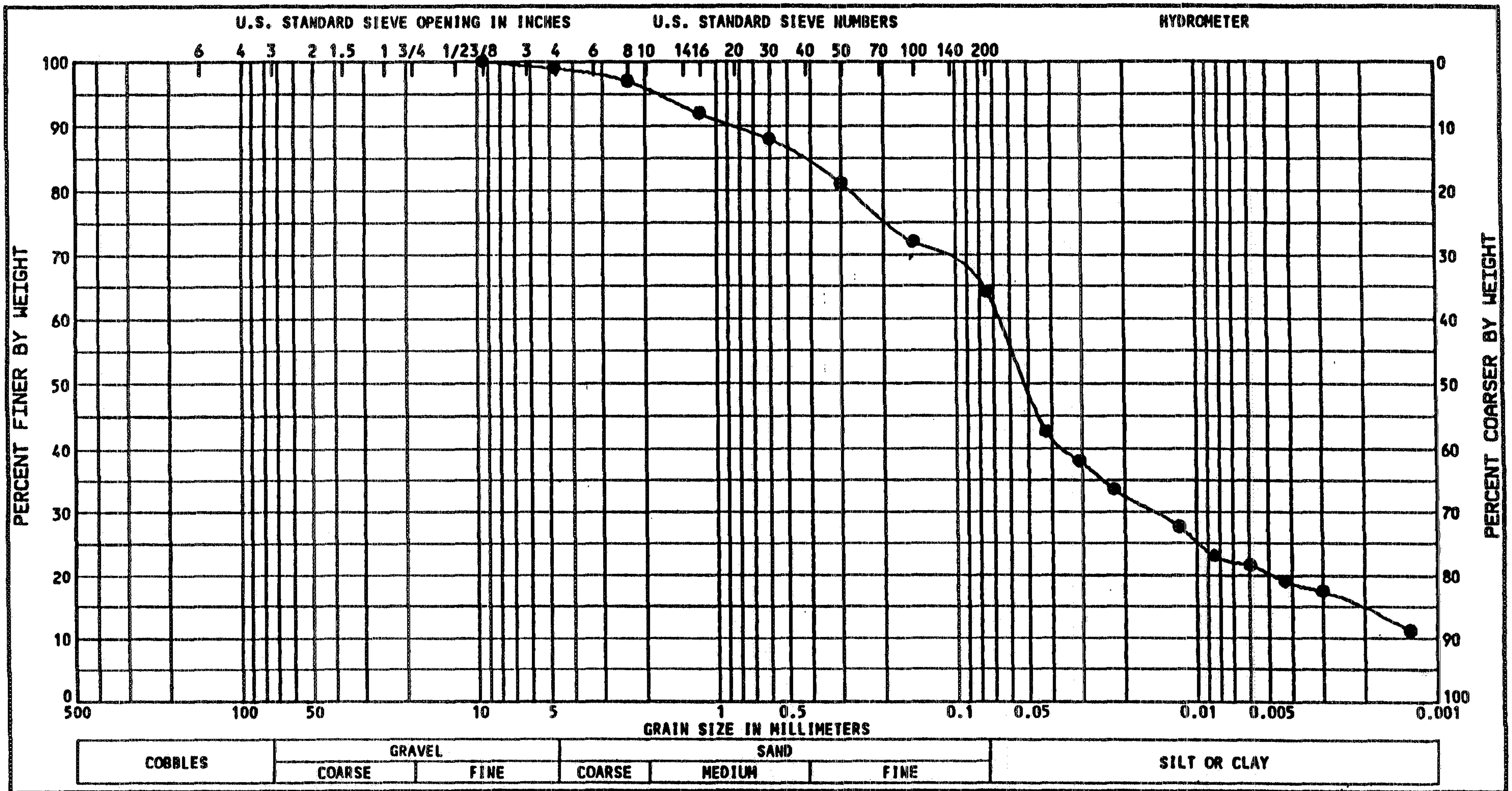


Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121314 @ 29.5	SANDY LEAN CLAY CL		29	16	13

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County
Area **GR-93-6**
Notes **F.S. 10**
Date **10/25/93**

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121351 @ 31.0	SANDY LEAN CLAY CL		25	14	11

Project Upper Guadalupe River
Santa Clara County

Area GR-93-6

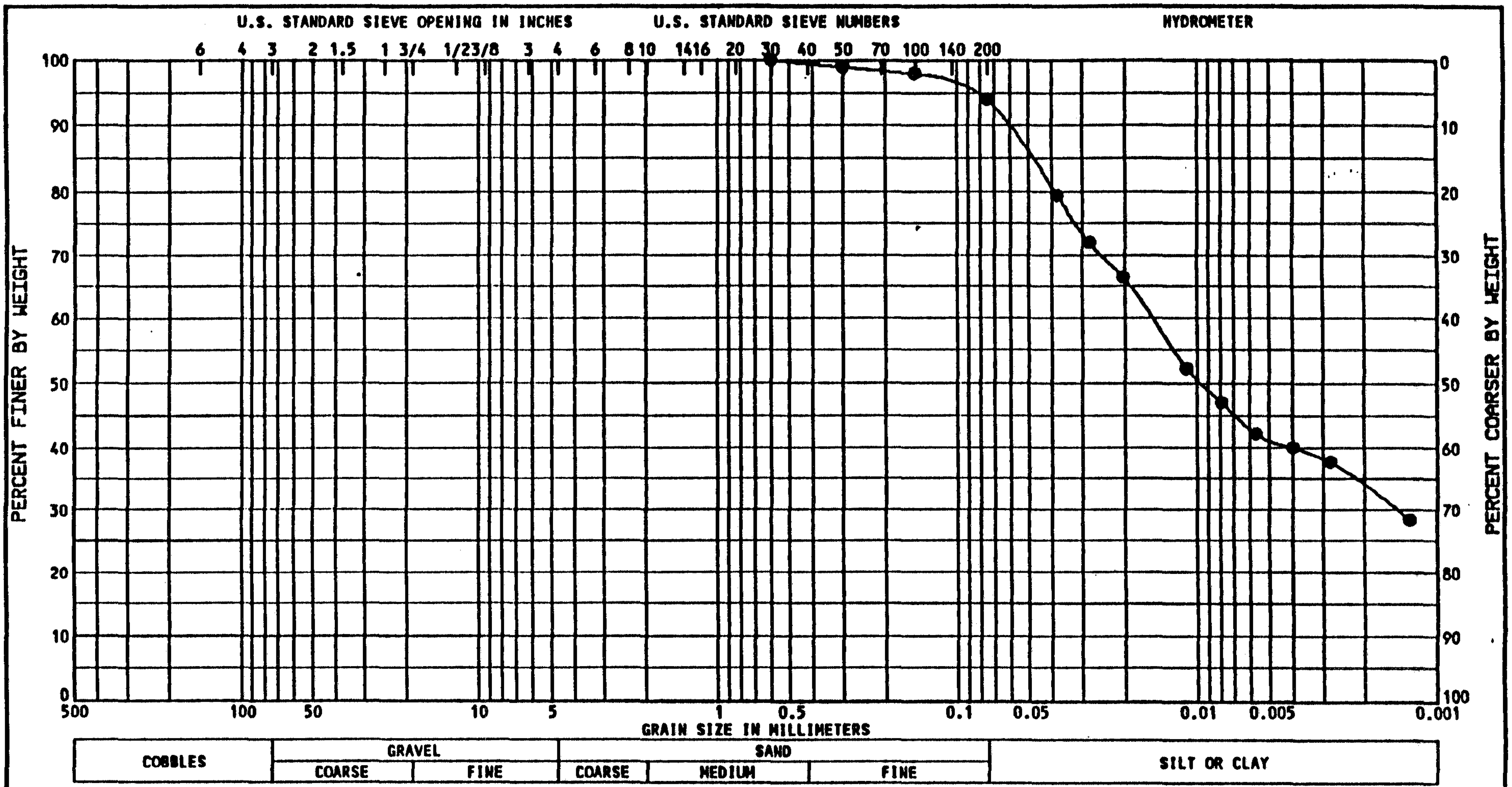
Notes F.S. 11

Date 10/27/93

GRADATION CURVES

292

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121324 @ 34.0	LEAN CLAY CL		38	13	25

Project **Upper Guadalupe River**
Santa Clara County

Area **GR-93-6**

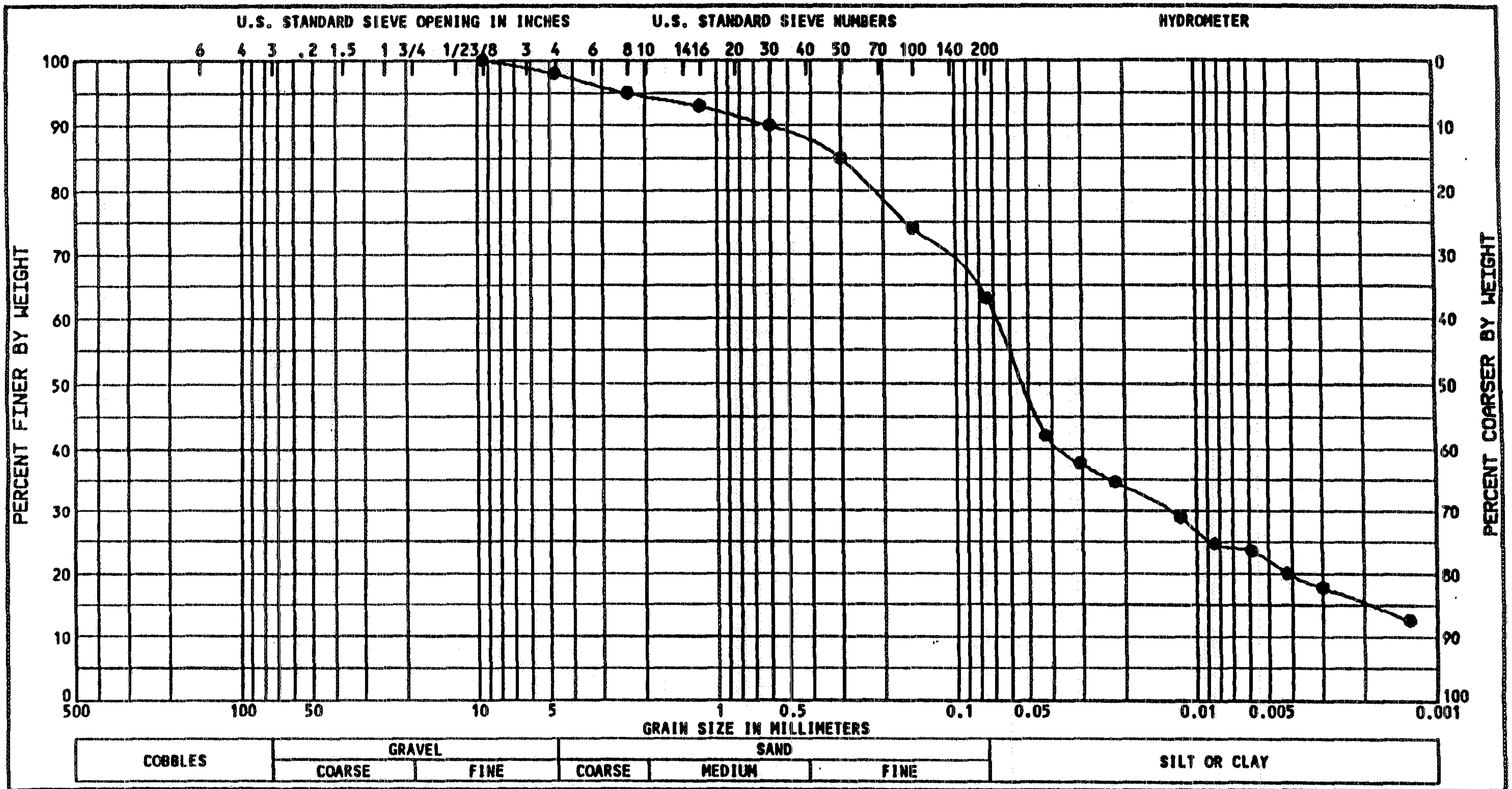
Notes **F.S. 12**

Date **10/27/93**

GRADATION CURVES

C63

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



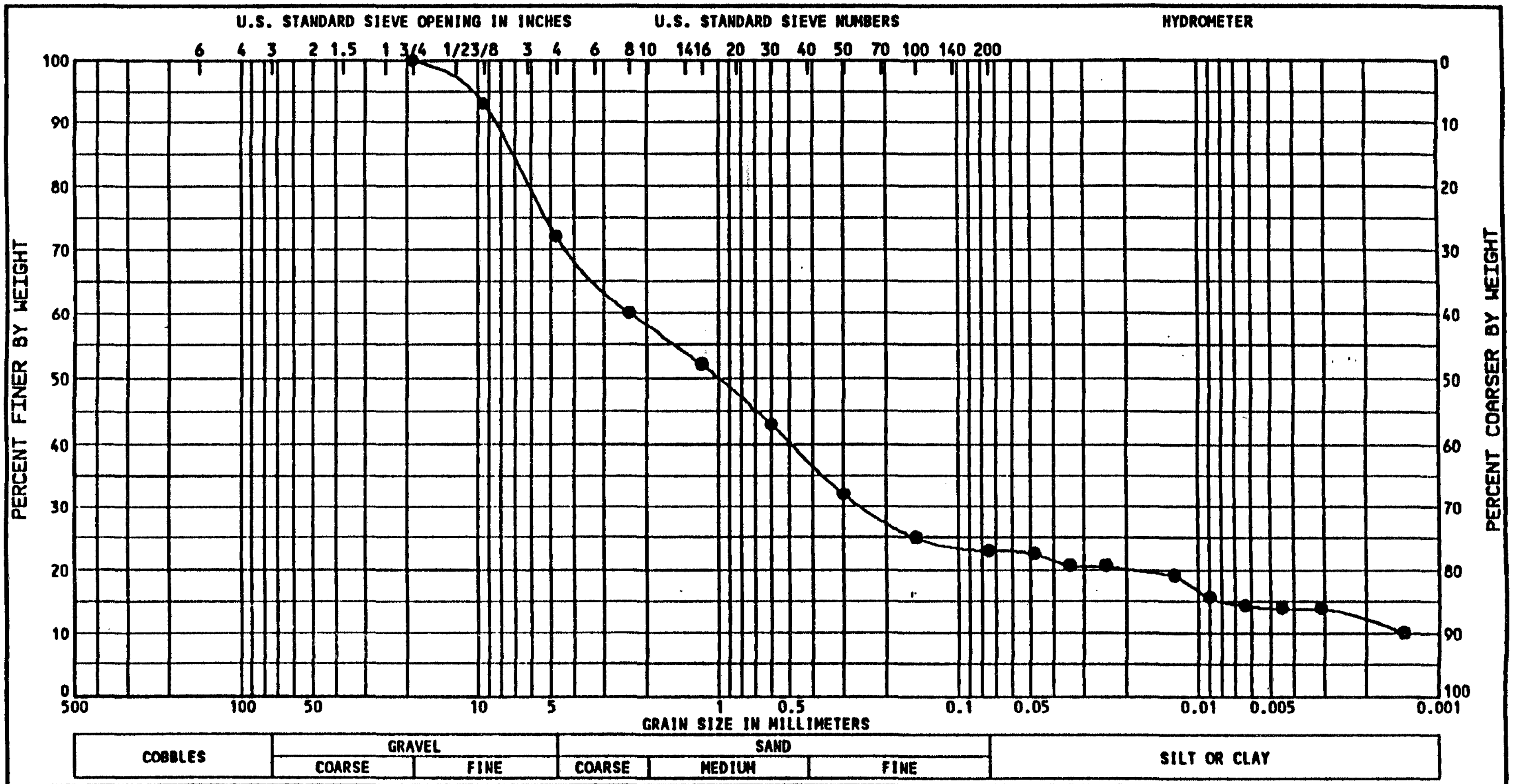
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121361 @ 6.5	SANDY LEAN CLAY CL		26	15	11

Project **Upper Guadalupe River**
Santa Clara County
Area **GR-93-7**
Notes **F.S. 2**
Date **10/27/93**

GRADATION CURVES

c64

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



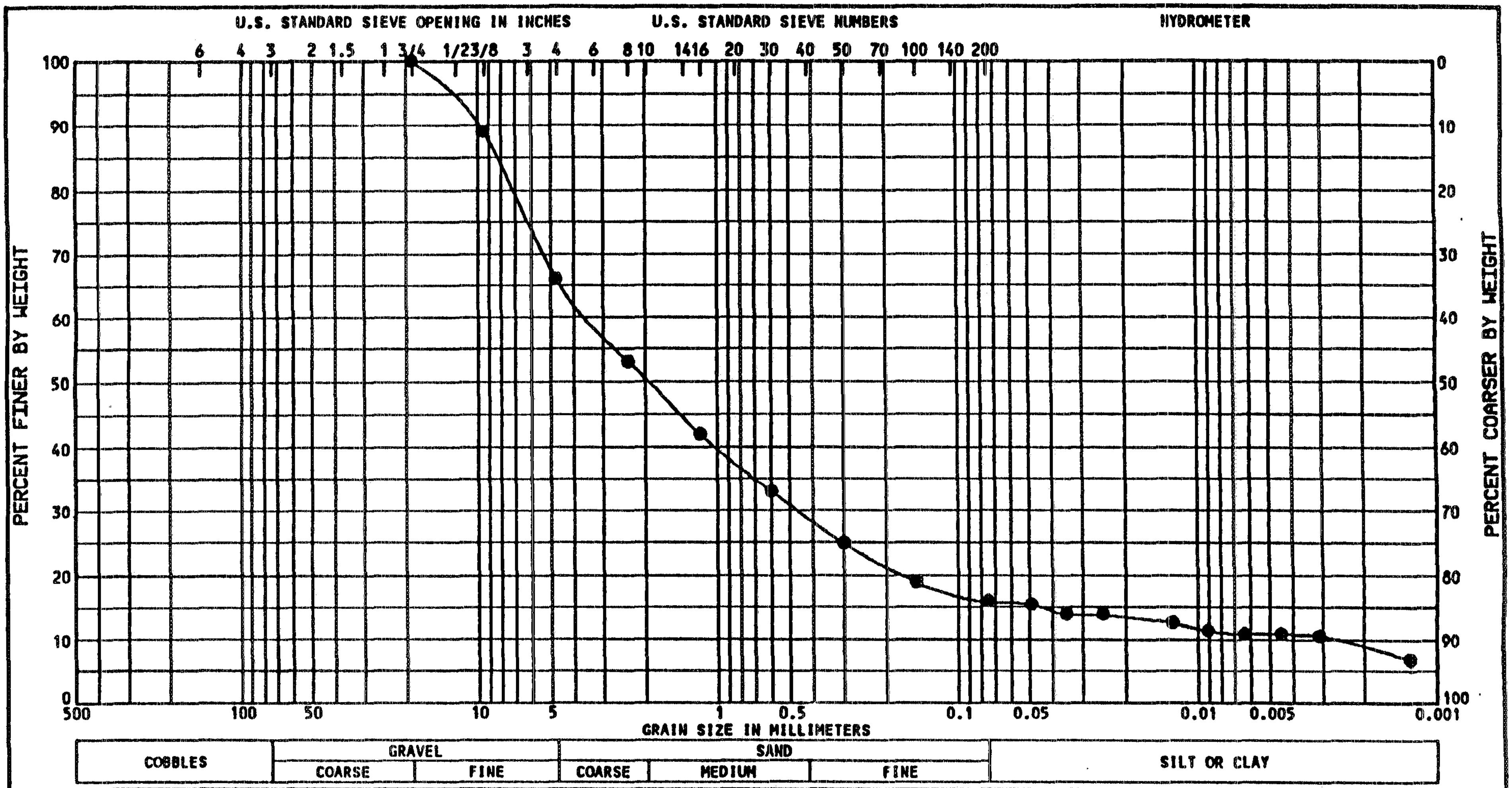
Specimen Identification	Classification	Nat W%	LL	PL	PI
● 121360 @ 14.0	CLAYEY SAND with GRAVEL SC		26	13	13

Project Upper Guadalupe River
Santa Clara County
Area GR-93-7
Notes F.S. 4
Date 10/26/93

GRADATION CURVES

C65

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121358 @ 17.0	CLAYEY SAND with GRAVEL SC		27	13	14

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County
Area **GR-93-7**
Notes **F.S. 6**
Date **10/26/93**

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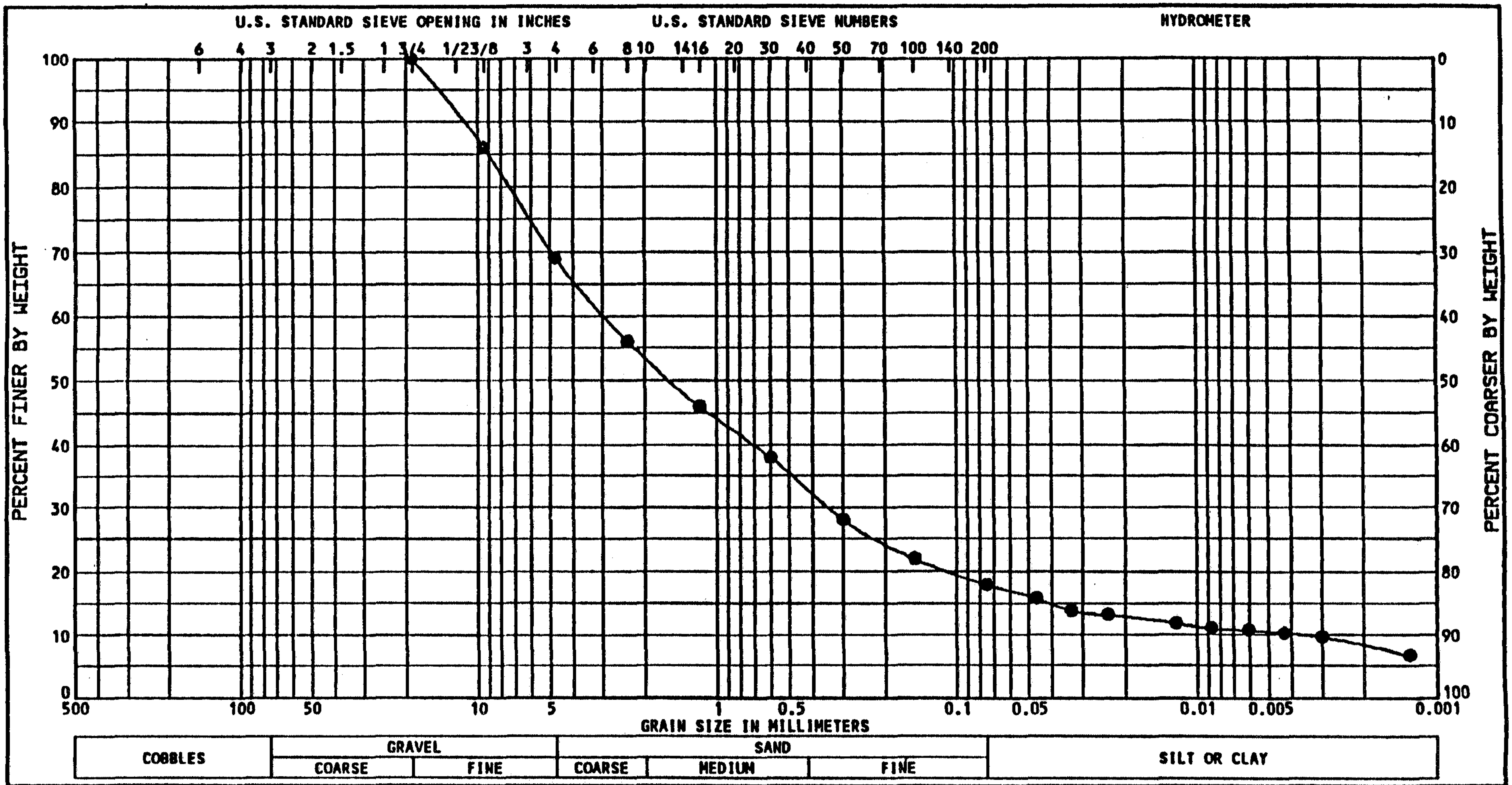

Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121355 @ 21.2	CLAYEY SAND with GRAVEL SC		28	12	16

Project **Upper Guadalupe River**
Santa Clara County
Area **GR-93-7**
Notes **F.S. 9**
Date **10/26/93**

GRADATION CURVES

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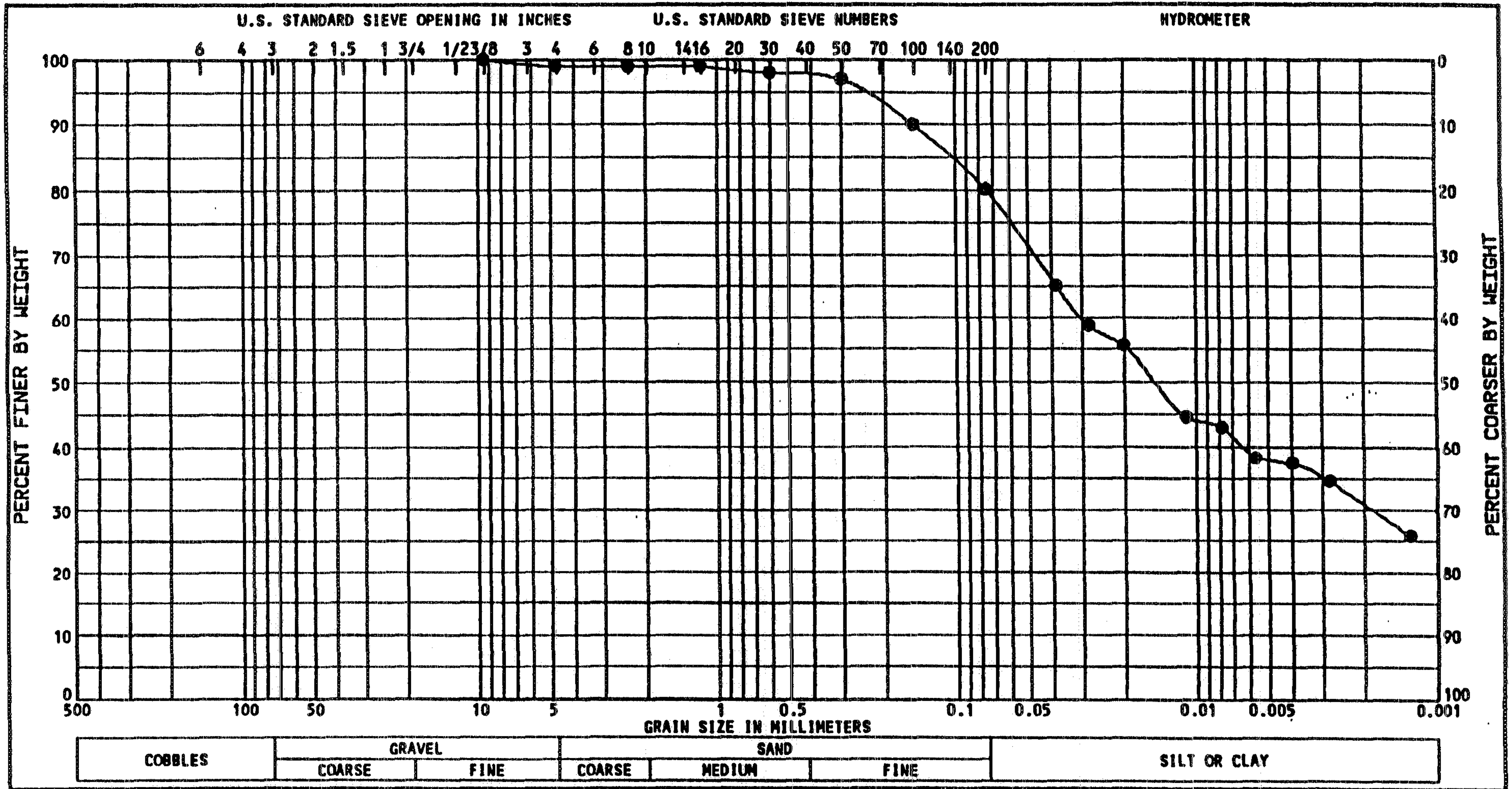


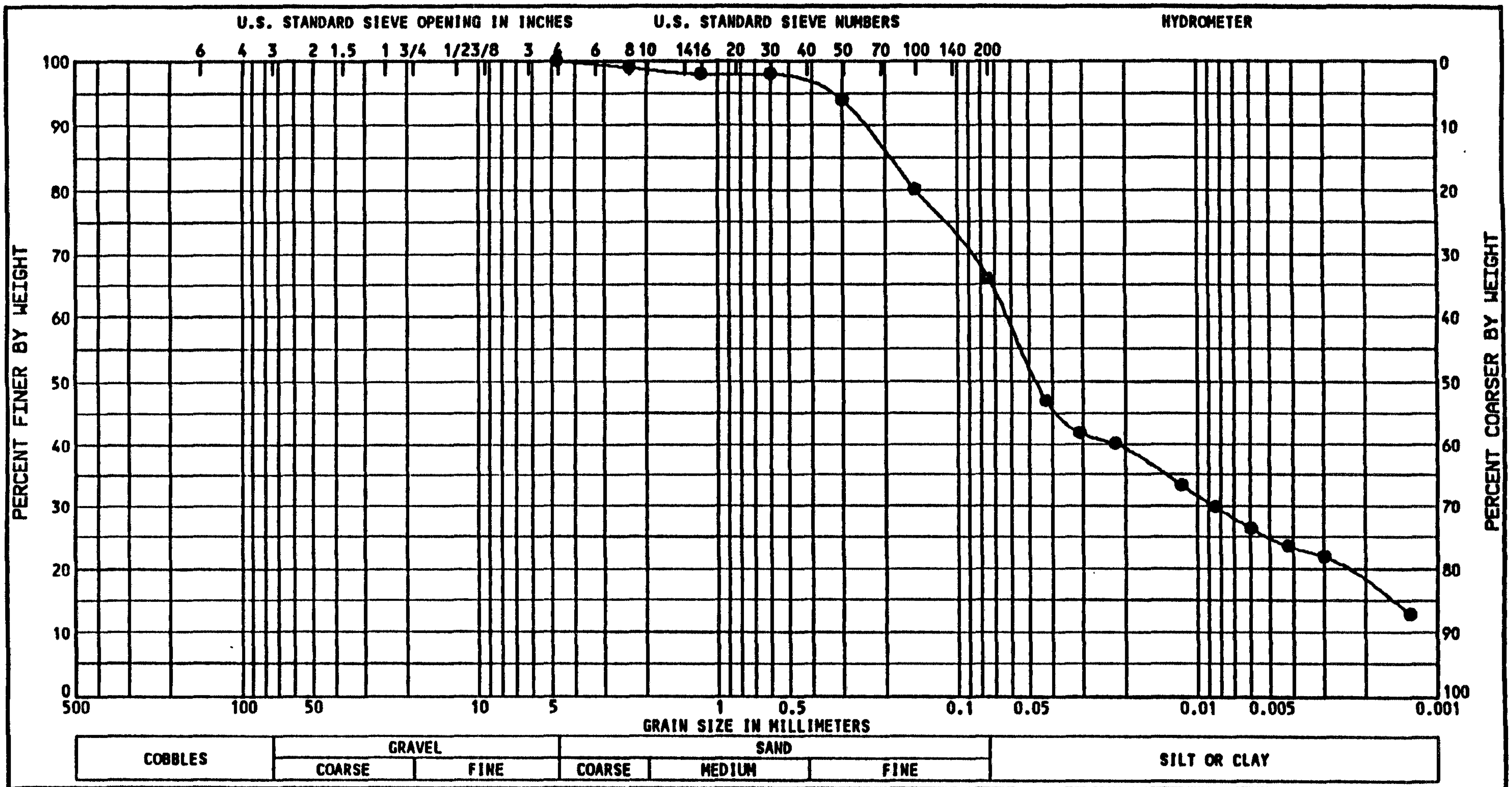
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121353 @ 26.0	LEAN CLAY with SAND CL		31	13	18

Project **Upper Guadalupe River**
Santa Clara County
Area **GR-93-7**
Notes **F.S. 12**
Date **10/27/93**

GRADATION CURVES

C68

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



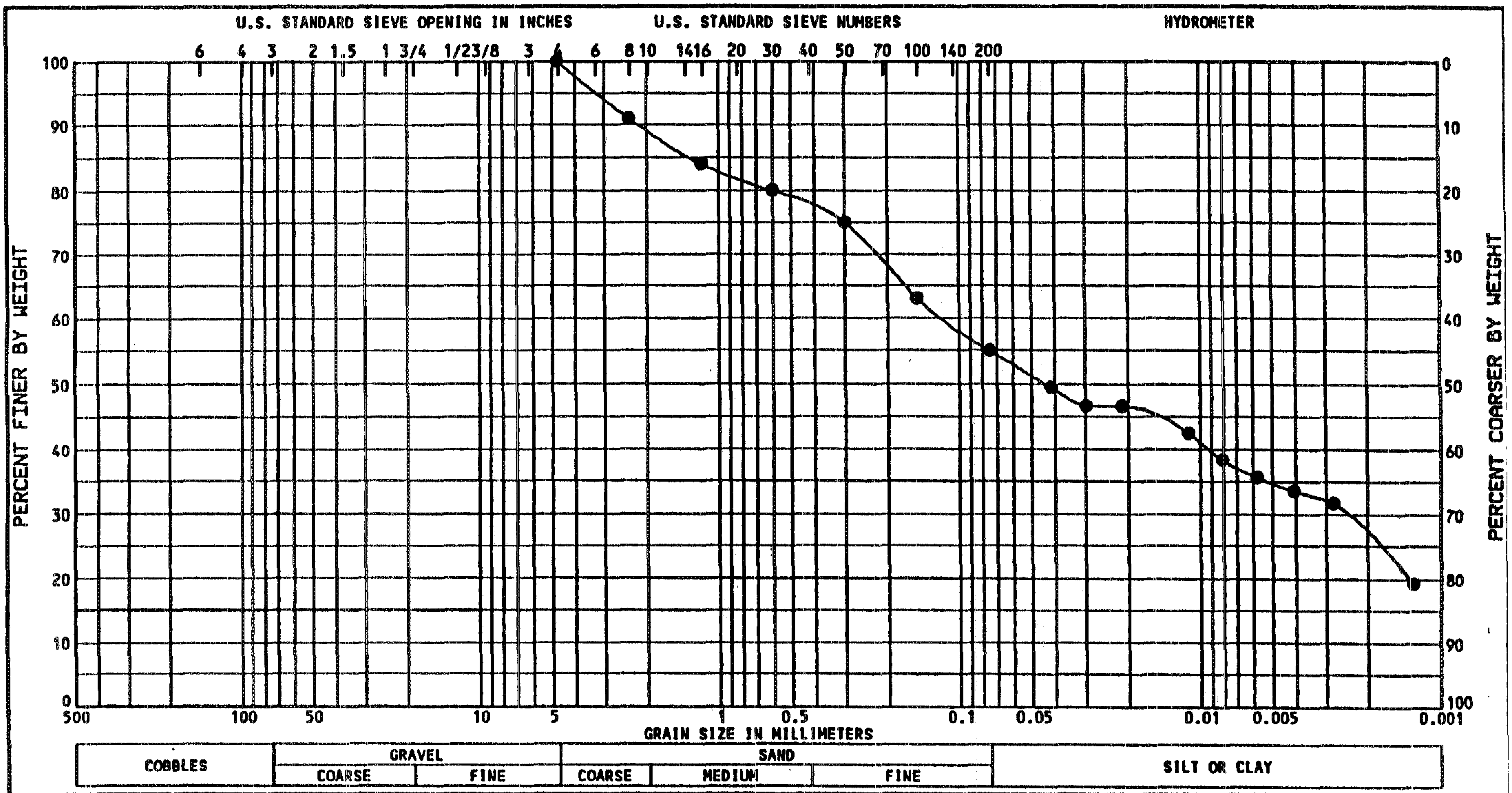
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121352 @ 29.0	SANDY LEAN CLAY CL		29	15	14

Project Upper Guadalupe River
Santa Clara County
Area GR-93-7
Notes F.S. 14
Date 10/27/93

GRADATION CURVES

692

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

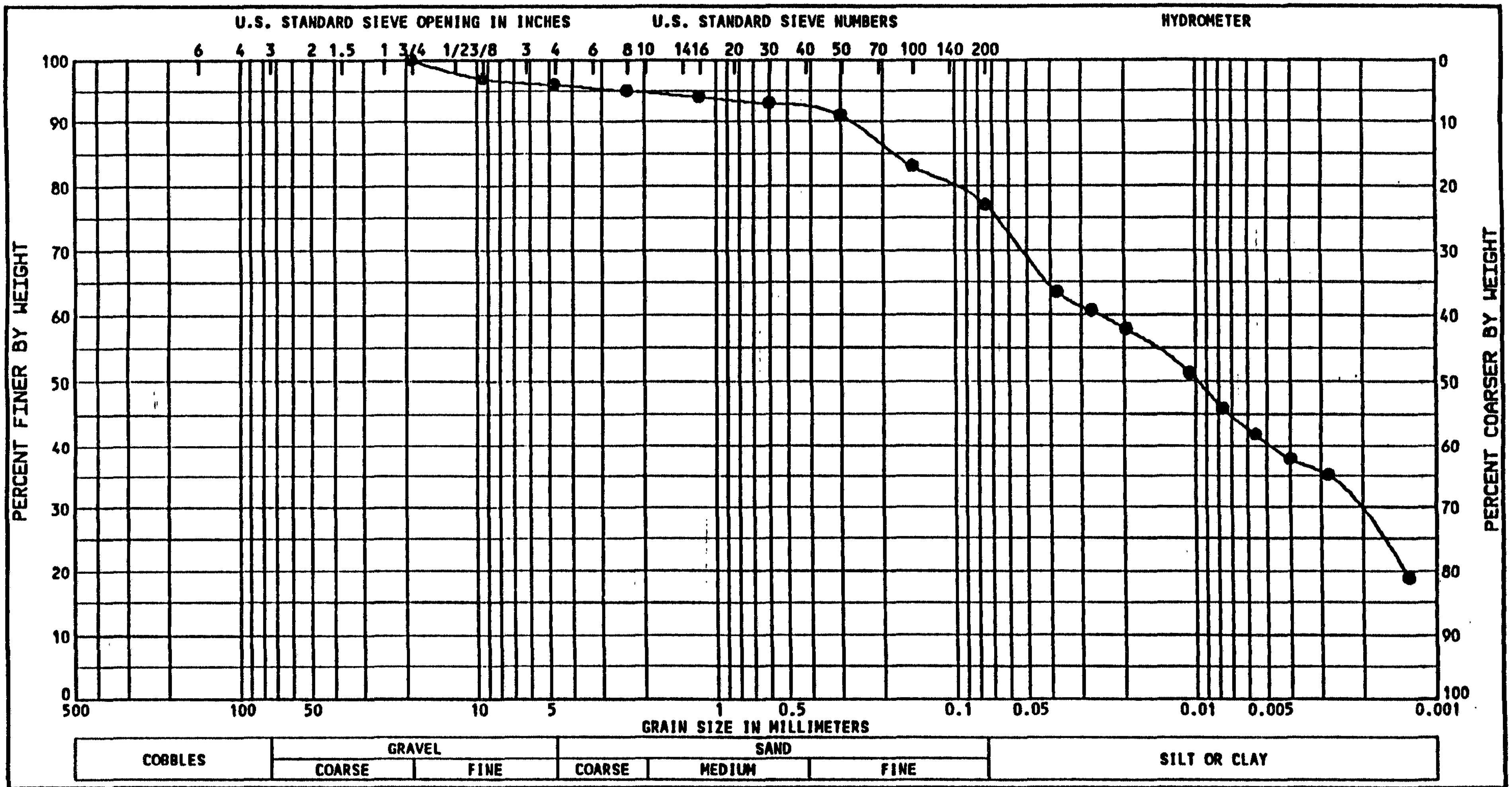


Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121294 @ 10.5	SANDY LEAN CLAY CL		31	13	18

Project Upper Guadalupe River
Santa Clara County
Area CC-93-1
Notes F.S. 3
Date 10/27/93

GRADATION CURVES

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



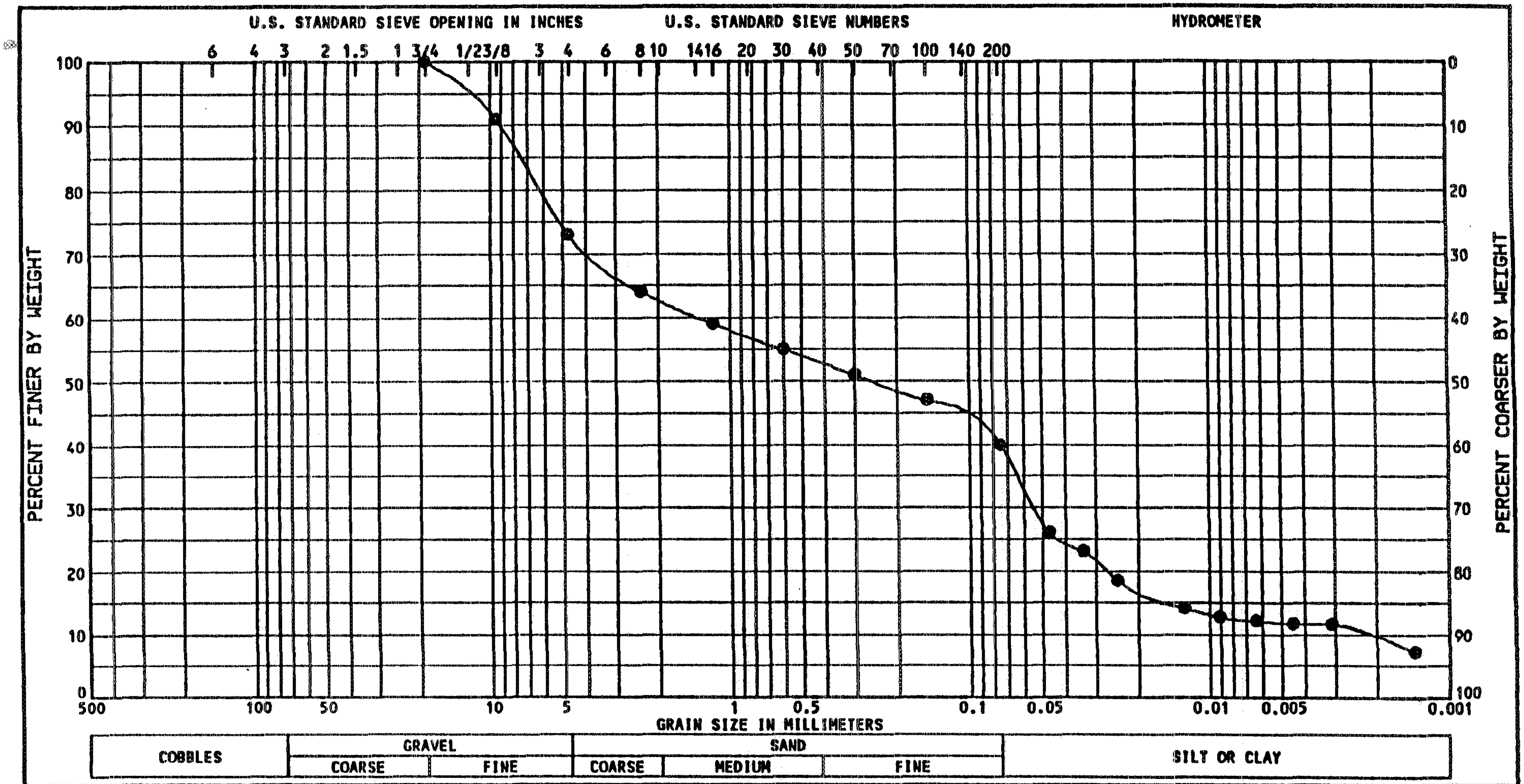
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121295 @ 20.5	LEAN CLAY with SAND CL		36	17	19

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County
Area **CC-93-1**
Notes **F.S. 5**
Date **10/27/93**

C71

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



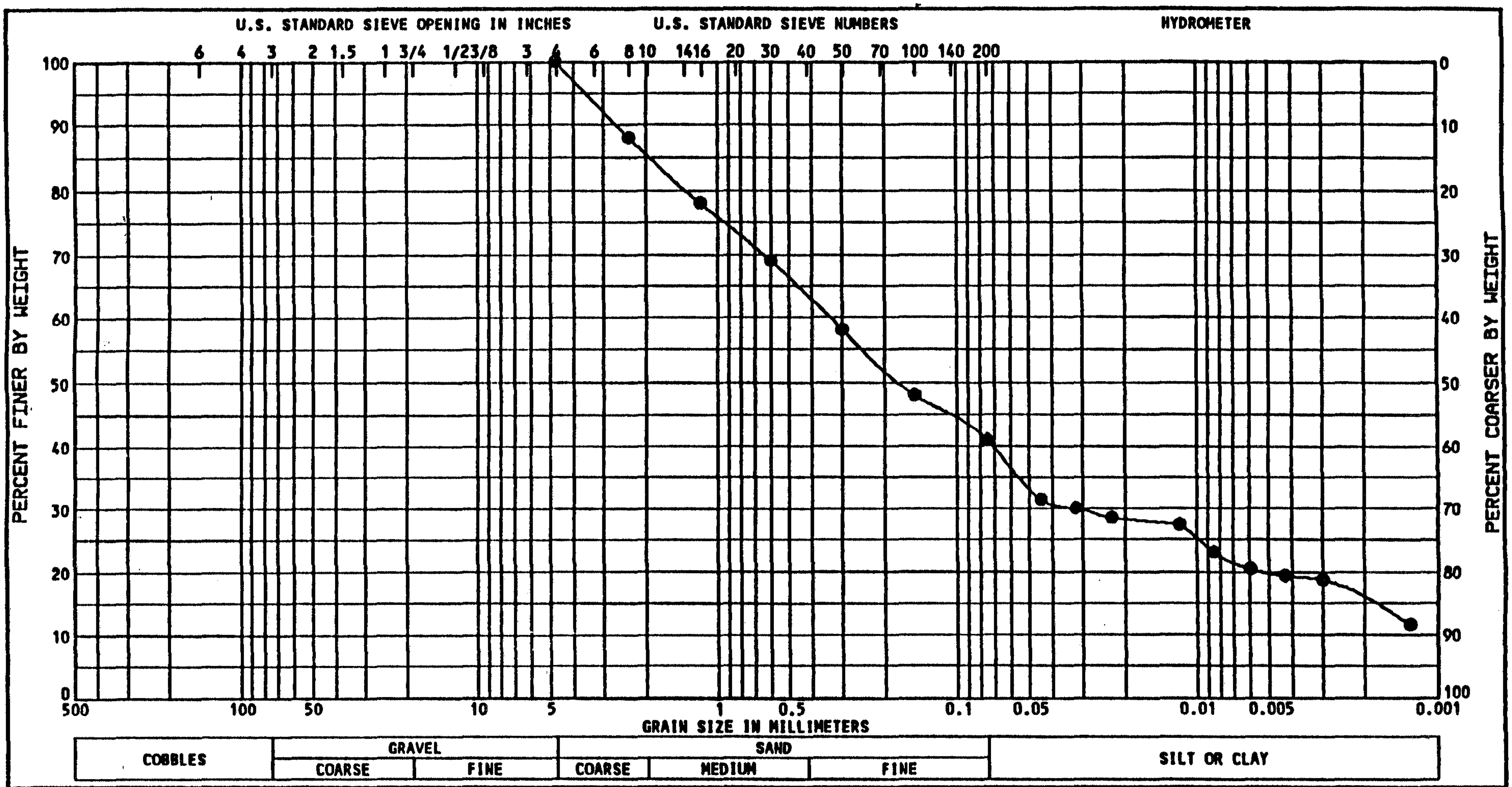
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121296 @ 17.5	CLAYEY SAND with GRAVEL SC		26	14	12

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County
Area **CC-93-2**
Notes **F.S. 7**
Date **10/26/93**

C72

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

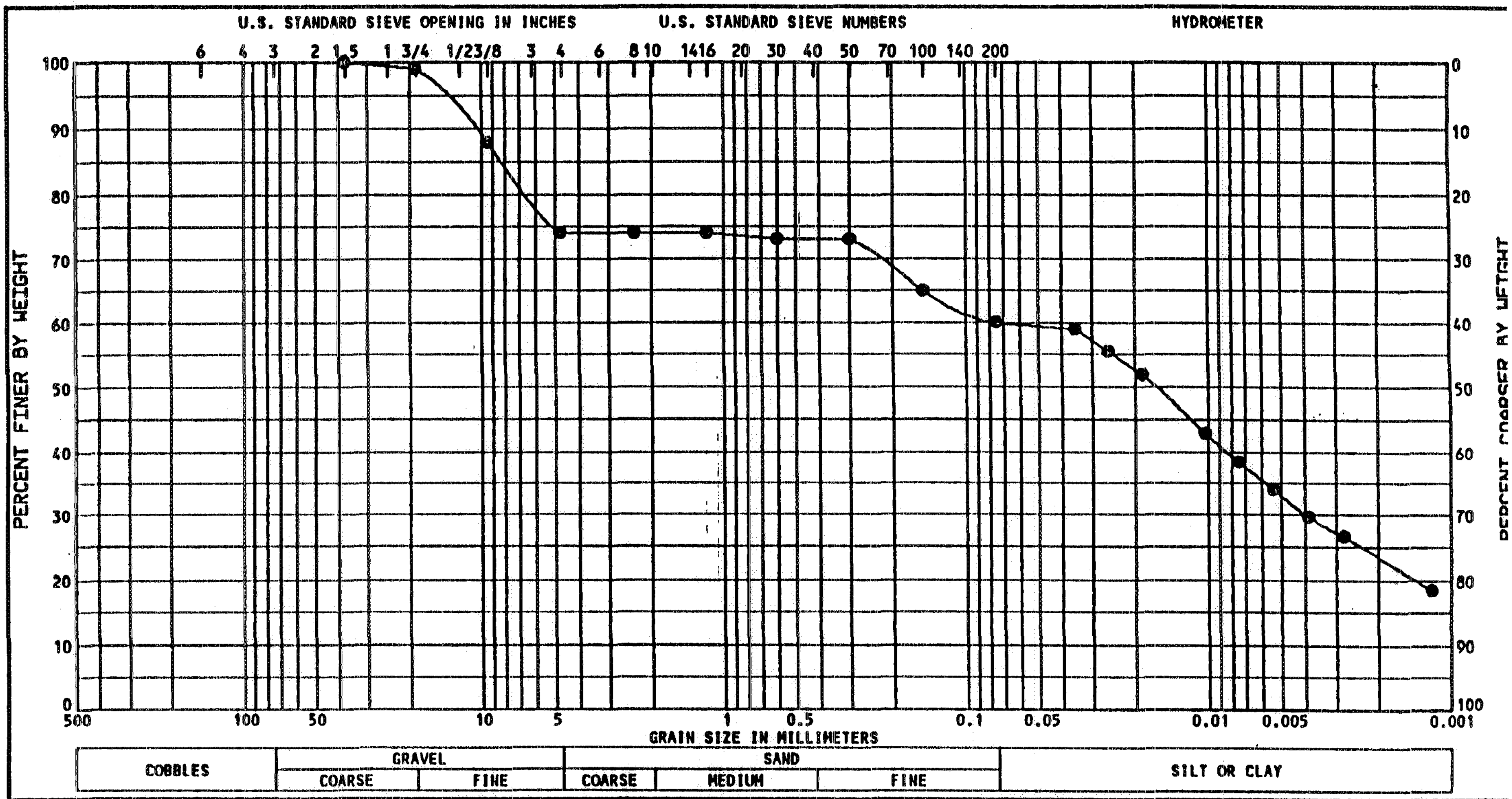


Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121393 @ 0.0	CLAYEY SAND SC		26	14	12

GRADATION CURVES

Project	**Upper Guadalupe River**
	Santa Clara County
Area	**RC-93-1**
Notes	**F.S. 1**
Date	**10/27/93**

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



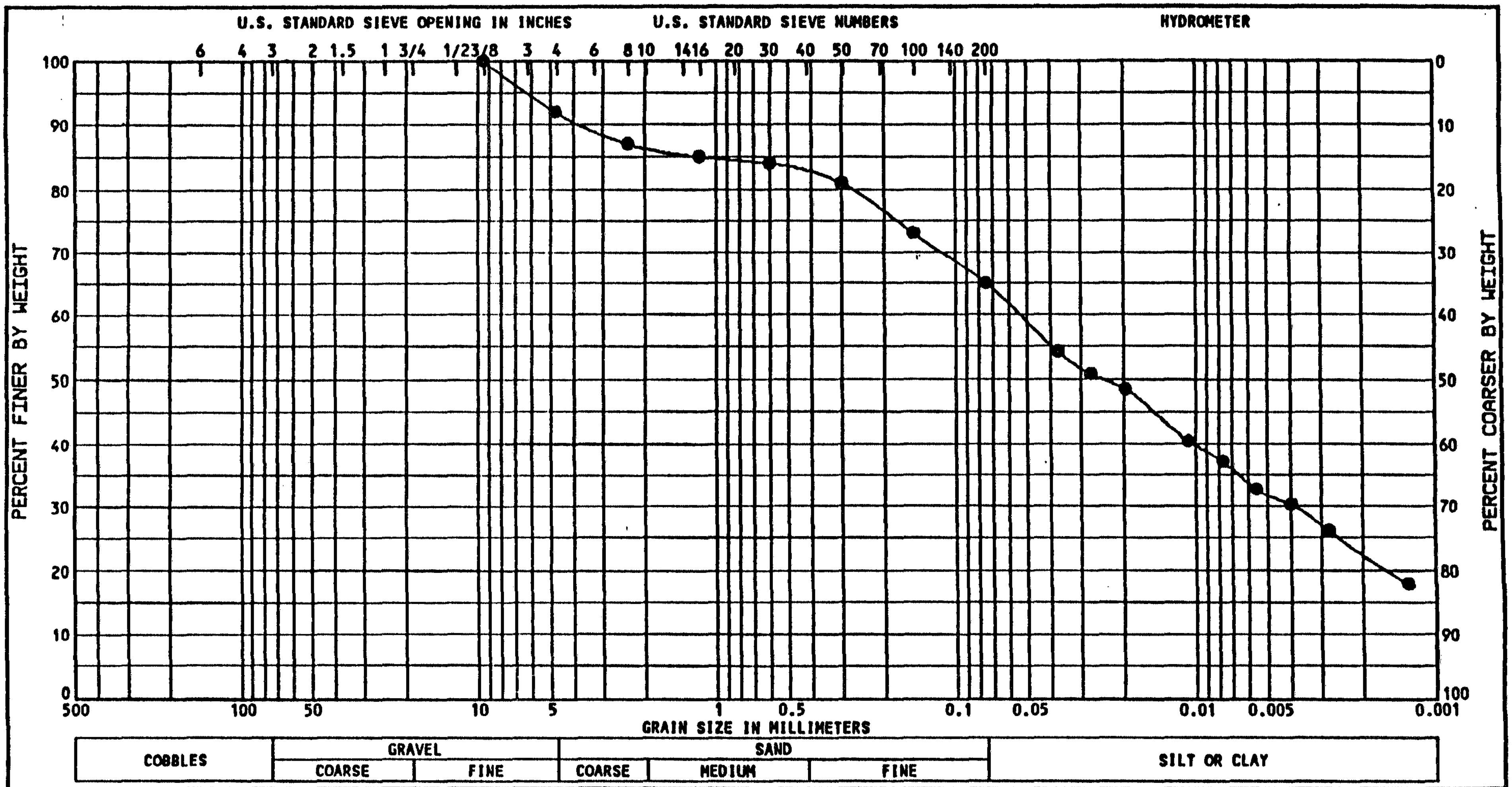
Specimen Identification	Classification	Nat w%	LL	PL	PI
⊙ 121394 @ 6.0	GRAVELLY LEAN CLAY CL		38	17	21

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County
Area **RC-93-1**
Notes **F.S. 3**
Date **10/27/93**

C74

DEP...TMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121371 @ 18.5	SANDY LEAN CLAY CL		29	14	15

GRADATION CURVES

Project Upper Guadalupe River
Santa Clara County

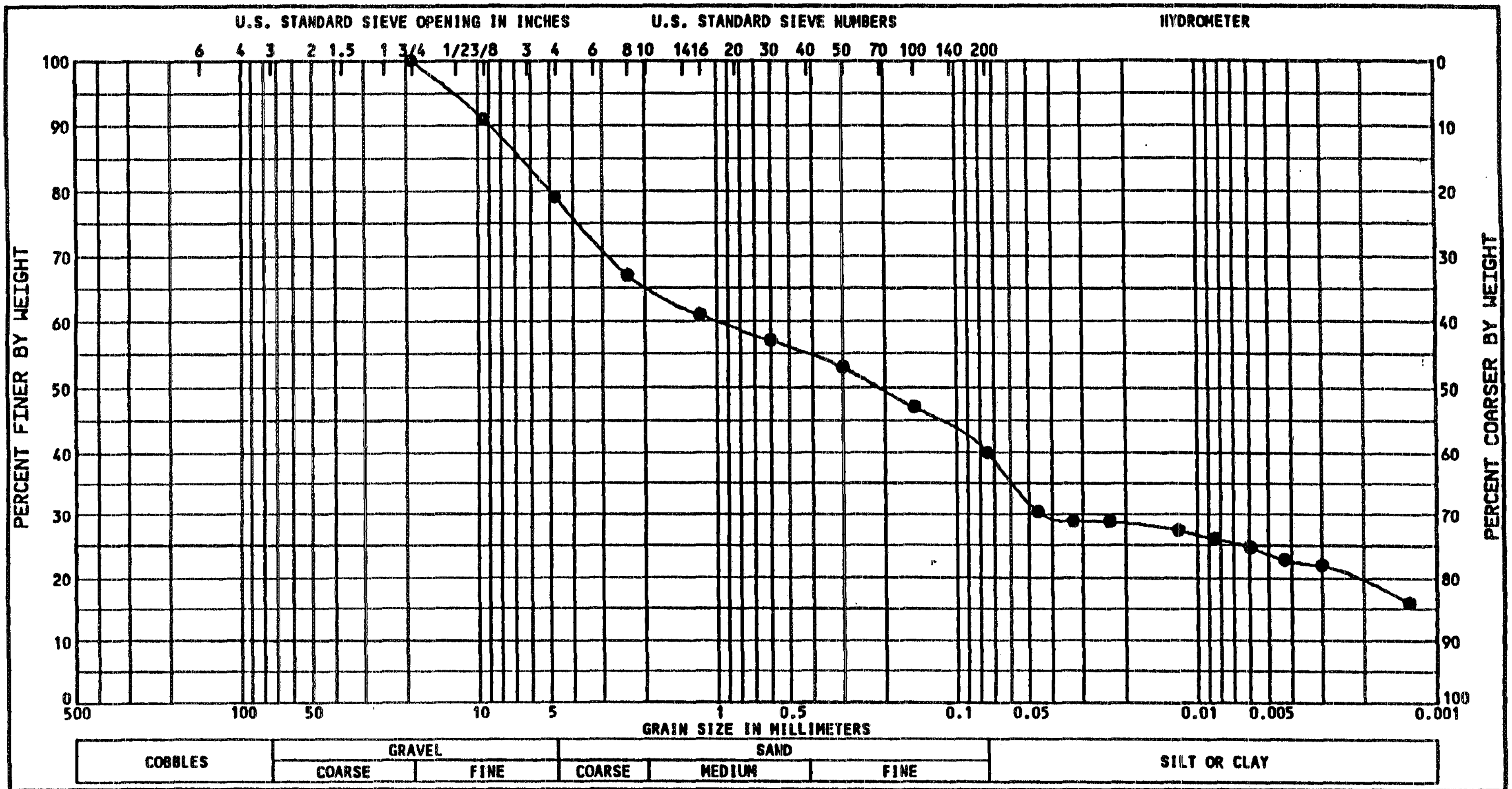
Area RC-93-1

Notes F.S. 6

Date 10/27/93

C75

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



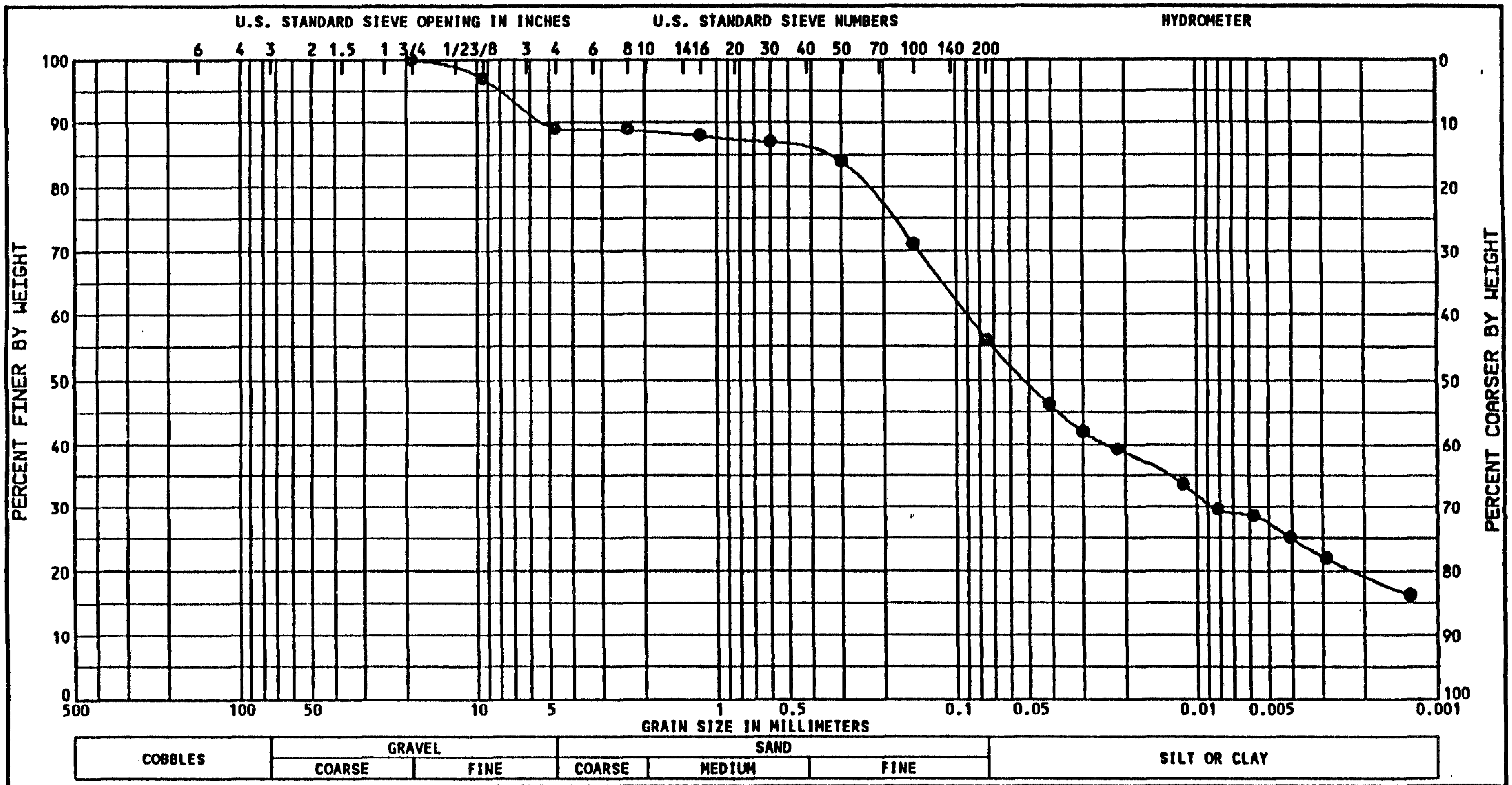
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121370 @ 20.5	CLAYEY SAND with GRAVEL SC		37	15	22

GRADATION CURVES

Project **Upper Guadalupe River**
Santa Clara County
Area **RC-93-1**
Notes **F.S. 8**
Date **10/27/93**

C76

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121368 @ 26.5	SANDY LEAN CLAY CL		30	13	17

Project **Upper Guadalupe River**
Santa Clara County

Area **RC-93-1**

Notes **F.S. 10**

Date **10/27/93**

GRADATION CURVES

DEPARTMENT OF THE ARMY
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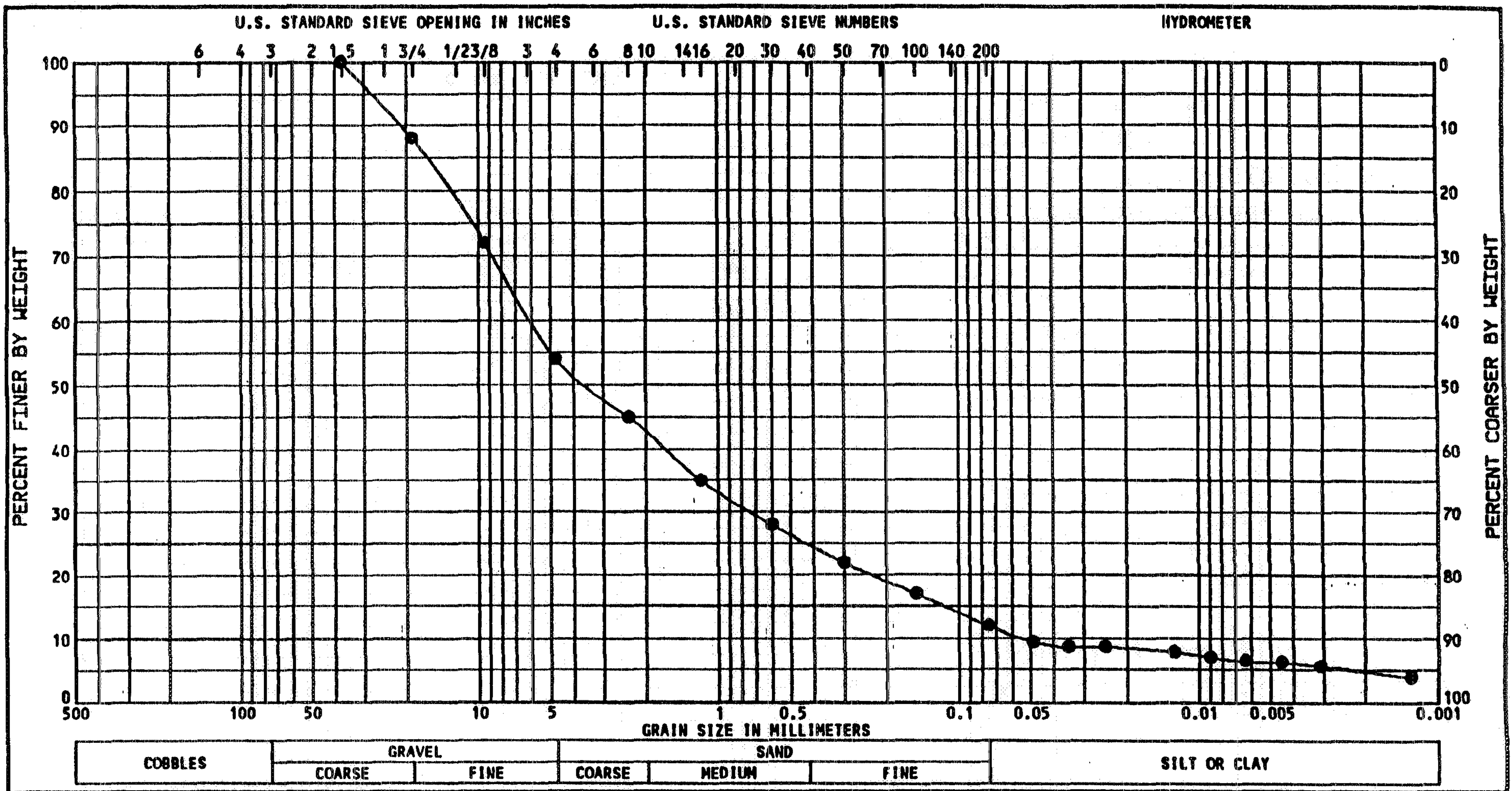


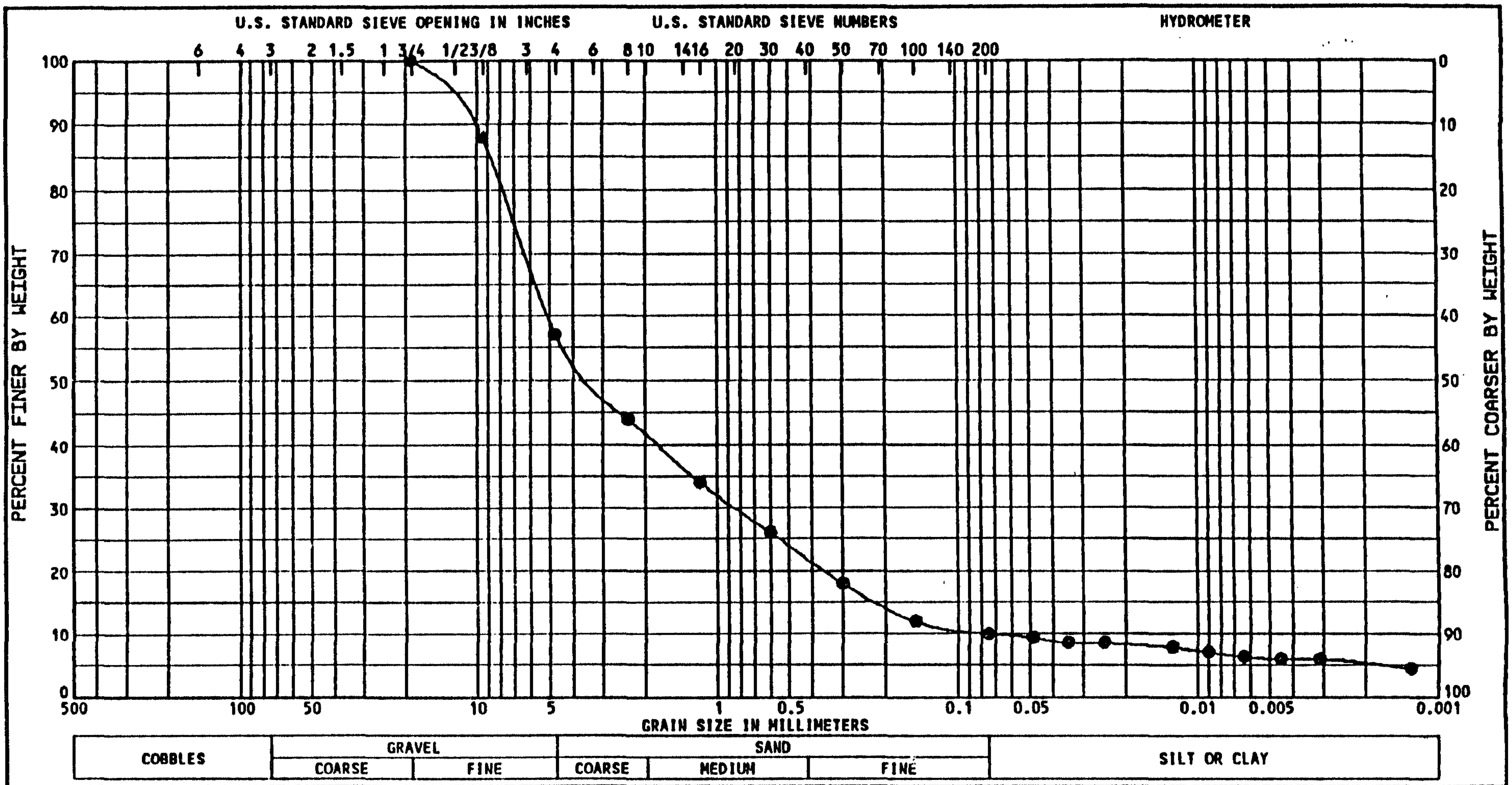
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121365 @ 34.0					

Project **Upper Guadalupe River**
Santa Clara County
Area **RC-93-1**
Notes **F.S. 13**
Date **10/26/93**

GRADATION CURVES

C78

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



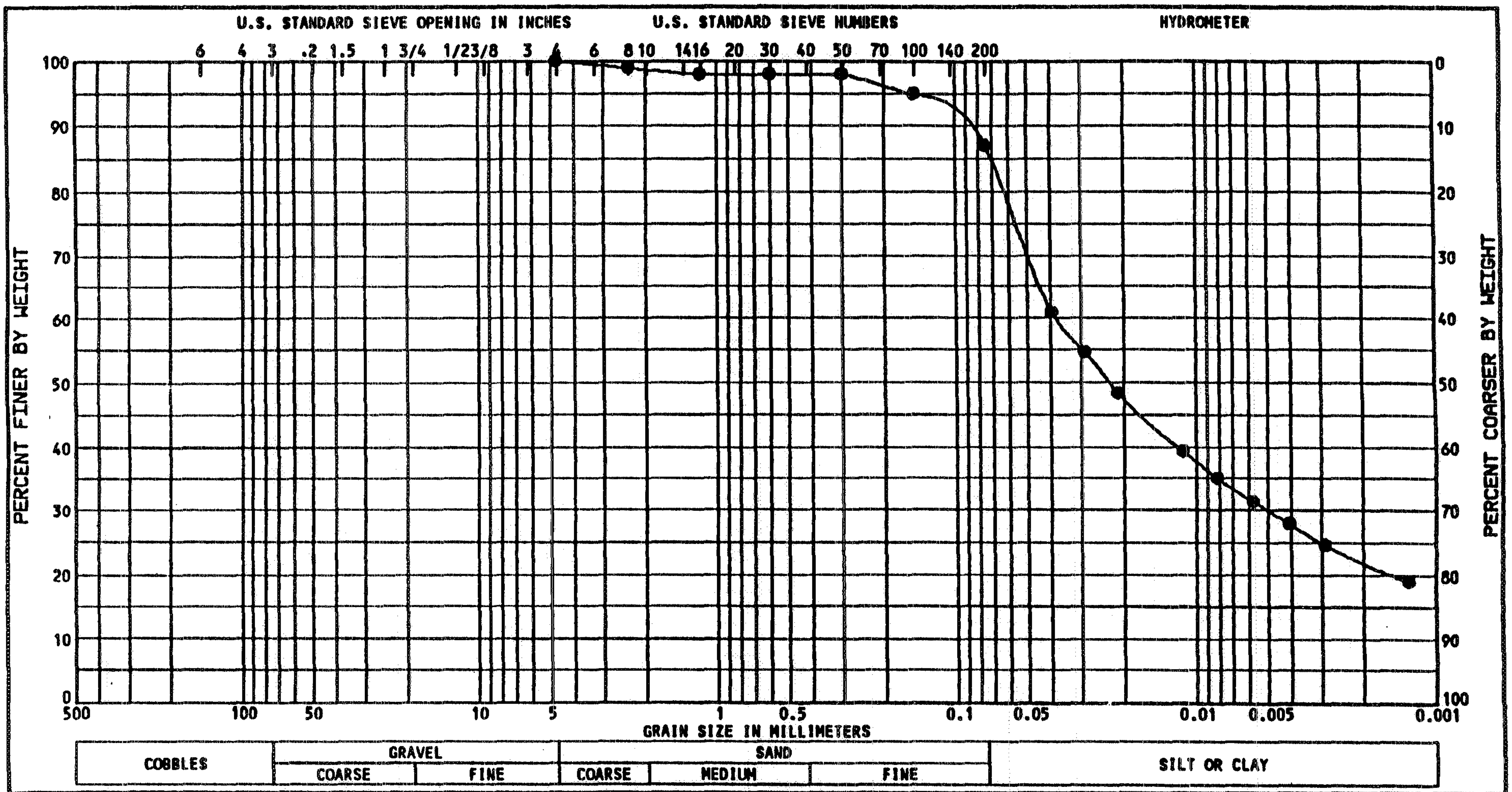
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121362 @ 37.0					

Project **Upper Guadalupe River**
Santa Clara County
Area **RC-93-1**
Notes **F.S.15,16**
Date **10/26/93**

GRADATION CURVES

c79

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121379 @ 9.5	LEAN CLAY CL		36	16	20

GRADATION CURVES

Project Upper Guadalupe River
Santa Clara County

Area RC-93-2

Notes F.S. 4

Date 10/27/93

080

DEPARTMENT OF THE ARMY
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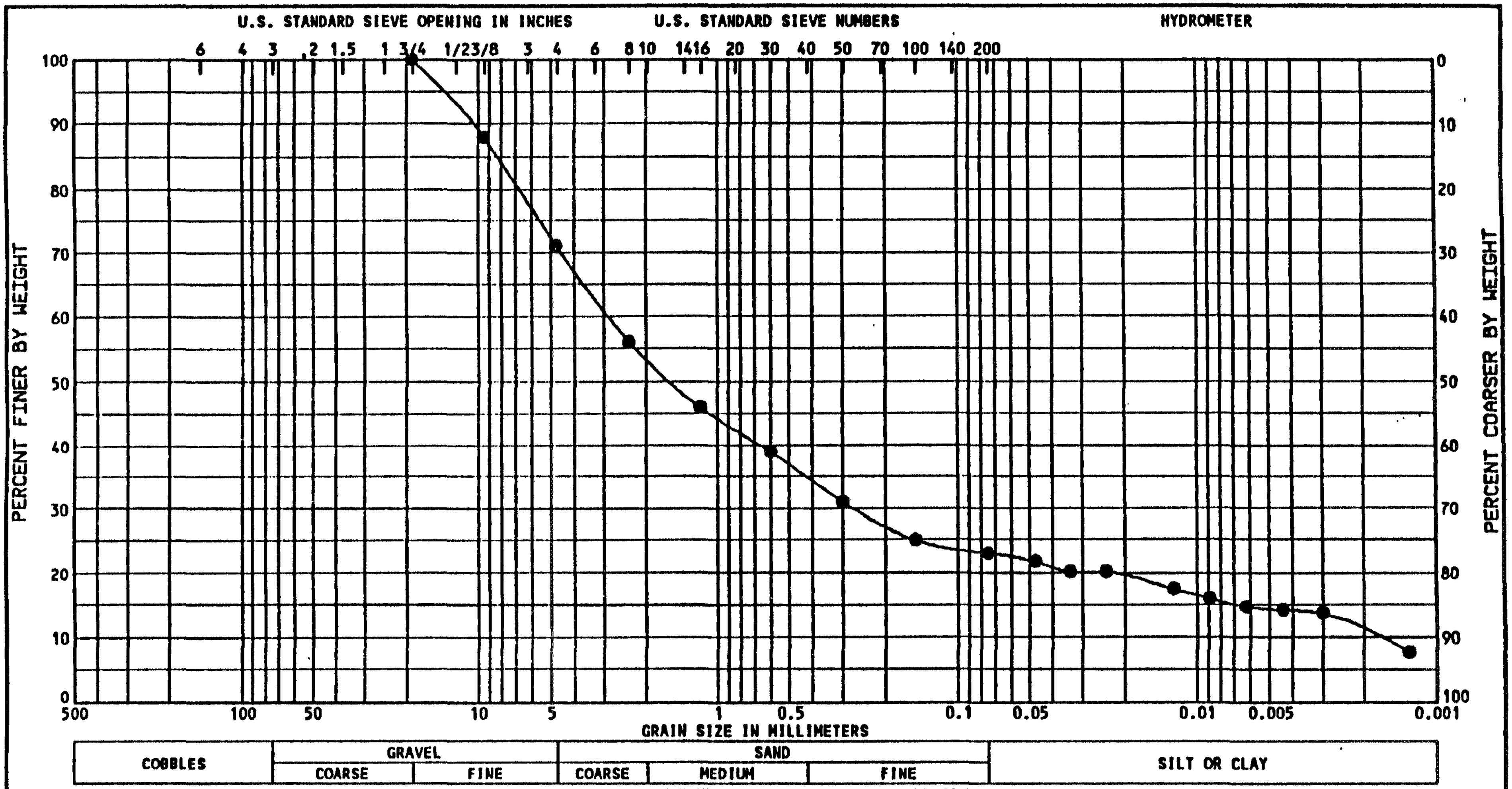


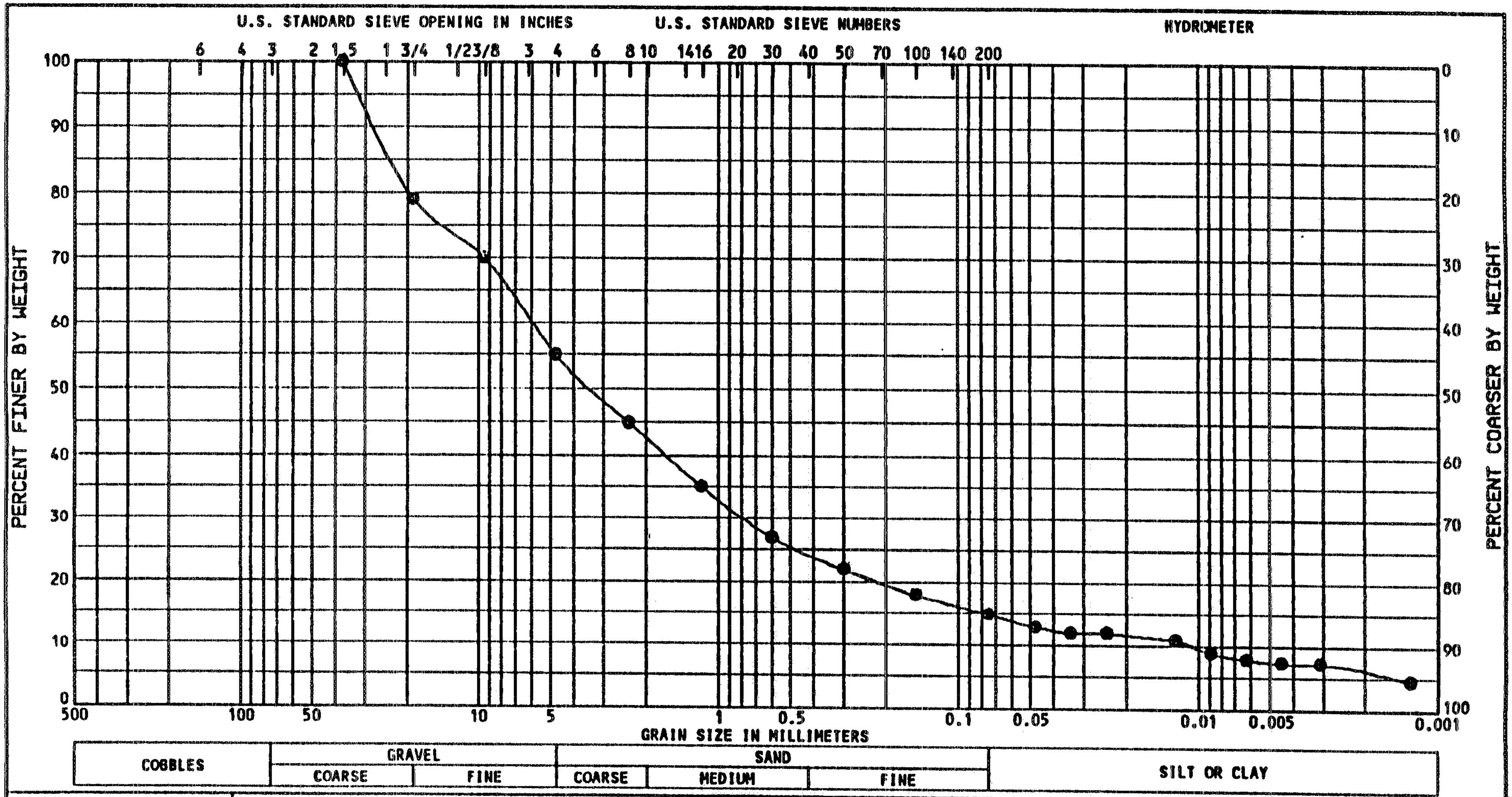
Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121376 @ 21.0	CLAYEY SAND with GRAVEL SC		32	13	19

Project Upper Guadalupe River
Santa Clara County
Area RC-93-2
Notes F.S. 7
Date 10/27/93

GRADATION CURVES

C81

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121374 @ 27.0	CLAYEY GRAVEL with SAND GC		39	13	26

Project Upper Guadalupe River
Santa Clara County

Area RC-93-2

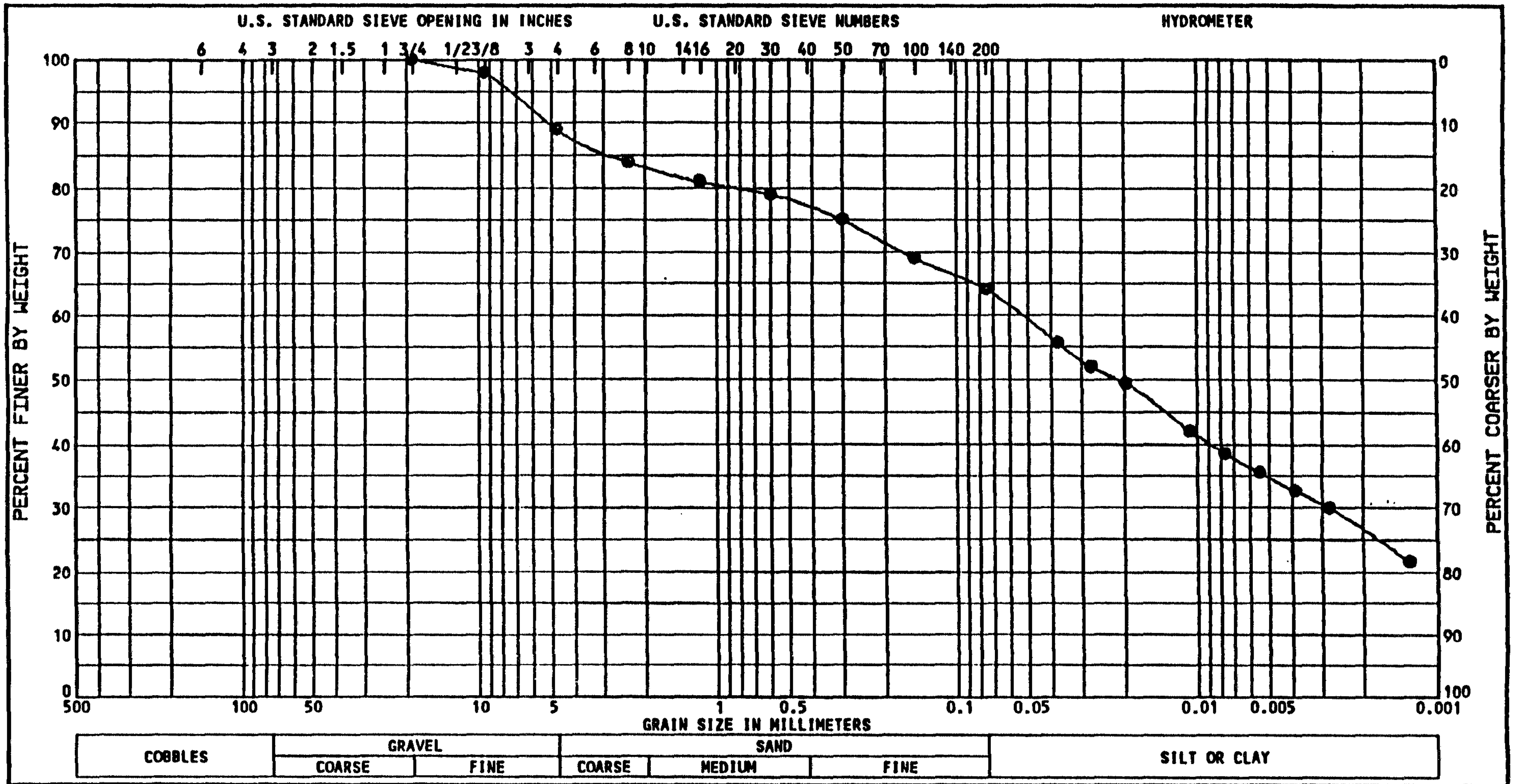
Notes F.S. 8,9

Date 10/27/93

GRADATION CURVES

C82

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



Specimen Identification	Classification	Nat w%	LL	PL	PI
● 121383 @ 8.0	SANDY LEAN CLAY CL		30	13	17

Project Upper Guadalupe River
Santa Clara County

Area RC-93-3

Notes F.S. 3

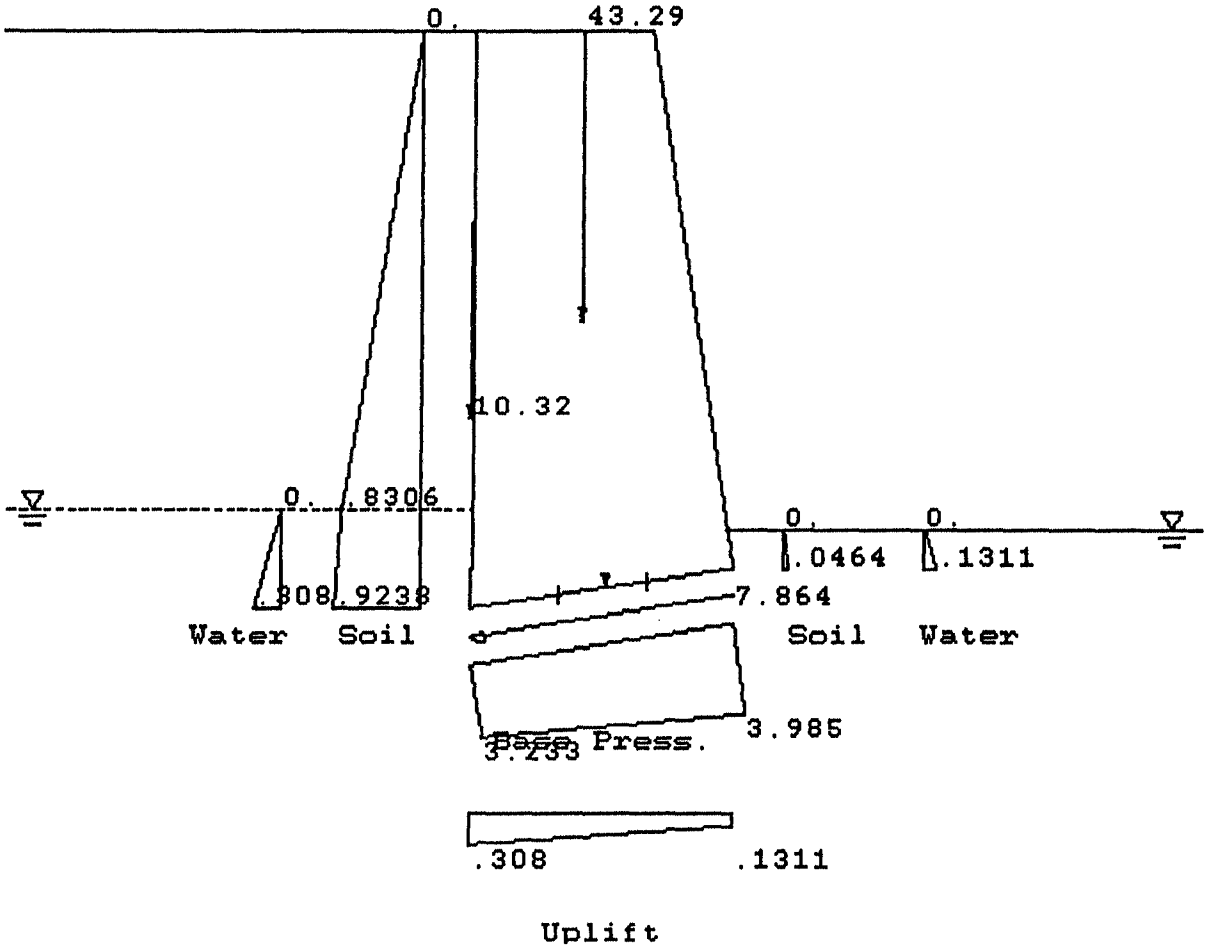
Date 10/27/93

GRADATION CURVES

C83

ATTACHMENT D

Crib Wall Sliding and Overturning Analyses


0.
43.29
10.32
0.
.8306
.308
.9238
Water
Soil
0.
.0464
0.
.1311
7.864
Soil
Water
3.985
3.233
Base Press.
.308
.1311
Uplift

```
****************** Echoprint of Input Data ******************

Date: 97/12/17                                     Time: 13.51.33

Structural geometry data:
  Elevation of top of stem (ELTS)           =    46.50 ft
  Height of stem (HTS)                      =    27.60 ft
  Thickness top of stem (TTS)               =     9.60 ft
  Thickness bottom of stem (TBS)            =    14.20 ft
  Dist. of batter at bot. of stem (TBSR)=         4.60 ft
  Depth of heel (THEEL)                     =     2.10 ft
  Distance of batter for heel (BTRH)        =      .00 ft
  Depth of toe (TTOE)                       =      .00 ft
  Width of toe (TWIDTH)                     =      .00 ft
  Distance of batter for toe (BTRT)         =      .00 ft
  Width of base (BWIDTH)                    =    14.30 ft
  Depth of key (HK)                         =      .00 ft
  Width of bottom of key (TK)               =      .00 ft
  Dist. of batter at bot. of key (BTRK) =          .00 ft

Structure coordinates:

   x (ft)    y (ft)
  ==================
       .00     16.80
       .00     18.90
       .10     18.90
       .10     46.50
      9.70     46.50
     14.30     18.90

NOTE: X=0 is located at the left-hand side
      of the structure.  The Y values correspond
      to the actual elevation used.

Structural property data:
  Unit weight of concrete =      .125 kcf

Driving side soil property data:

                          Moist    Saturated          Elev.
   Phi         c        Unit wt.   unit wt.   Delta   soil
  (deg)      (ksf)       (kcf)      (kcf)     (deg)   (ft)
=======================================================
   35.00      .000       .115        .120     35.00   46.50
```

```
Driving side soil geometry:

  Soil      Batter    Distance
  point    (in:1ft)     (ft)
  =============================
   1          .00    1000.00
   2          .00        .00
   3          .00     500.00

Driving side soil profile:

  Soil         x          y
  point       (ft)       (ft)
  =============================
   1     -1999.90      46.50
   2          .10      46.50

Resisting side soil property data:

                      Moist   Saturated  Elev.
   Phi         c     Unit wt.  unit wt.  soil     Batter
  (deg)      (ksf)    (kcf)     (kcf)    (ft)    (in:1ft)
========================================================
   35.00      .000      .115      .120   20.90      .00

Resisting side soil profile:

  Soil         x          y
  point       (ft)       (ft)
  =============================
   1         13.97      20.90
   2        513.97      20.90

Foundation property data:
  phi for soil-structure interface =    35.00 (deg)
  c for soil-structure interface   =     .000 (ksf)
  phi for soil-soil interface      =    35.00 (deg)
  c for soil-soil interface        =     .000 (ksf)

Water data:
  Driving side elevation   =    22.00 ft
  Resisting side elevation =    20.90 ft
  Unit weight of water     =    .0624 kcf
  Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
  Sliding FS    =    1.50
  Overturning   =  100.00% base in compression

Crack options:
  o  Crack depth is to be calculated
  o  Computed cracks *will* be filled with water

Strength mobilization factor =    .6667
```

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

```
***** Summary of Results *****

***************     *** Satisfied ***
* Overturning *     Required base in comp. = 100.00 %
***************     Actual base in comp.   = 100.00 %
                    Overturning ratio      =   3.56

Xr (measured from toe) =   6.98 ft
Resultant ratio        =  .4826
Stem ratio             =  .0000
Base pressure at heel =    3.2333 ksf
Base pressure at toe  =    3.9852 ksf

***********    *** Satisfied ***
* Sliding *    Min. Required =   1.50
***********    Actual FS     =   2.26
```

```
********************** Output Results **********************

Date: 97/12/17                                        Time: 13.51.33

***************************
**  Overturning Results  **
***************************

Solution converged in  1 iterations.

SMF used to calculate K's =    .6667
Alpha for the SMF         = -49.0954
Calculated earth pressure coefficients:
  Driving side at rest K    =    .2948
  Driving side at rest Kc   =    .5001
  Resisting side at rest K  =    .4264
  Resisting side at rest Kc =    .6530
  At-rest K's for resisting side calculated.

Depth of cracking =      .00 ft

** Driving side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
        22.00       .0000
        16.80       .3080

  Earth pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
        46.50       .0000
        22.00       .8306
        16.80       .9238

** Resisting side pressures **

  Water pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
        20.90       .0000
        18.90       .1311

  Earth pressures:
    Elevation   Pressure
      (ft)        (ksf)
    ======================
        20.90       .0000
        18.90       .0464
```

```
** Uplift pressures **

  Water pressures:
     x-coord.   Pressure
      (ft)        (ksf)
   ======================
         .00       .3080
       14.30       .1311

** Forces and moments **

=====================================================================
                 Part            |    Force (kips)    | Mom. Arm  |  Momen
                                 |  Vert.   |  Horiz.|   (ft)    |  (ft-k
=====================================================================
Structure:
  Structure weight...........    42.932                   -8.24    -353.5
Structure, driving side:
  Moist soil.................      .282                  -14.25      -4.0
  Saturated soil.............      .037                  -14.25       -.5
  Water above structure......      .000                     .00        .0
  Water above soil...........      .000                     .00        .0
  External vertical loads....      .000                     .00        .0
  Ext. horz. pressure loads..                  .000         .00        .0
  Ext. horz. line loads......                  .000         .00        .0
Structure, resisting side:
  Moist soil.................      .000                     .00        .0
  Saturated soil.............      .040                    -.11        .0
  Water above structure......      .000                     .00        .0
  Water above soil...........      .000                     .00        .0
Driving side:
  Effective earth loads......                14.737        7.92     116.7
  Shear (due to delta).......    10.319                  -14.30    -147.5
  Horiz. surcharge effects...                  .000         .00        .0
  Water loads................                  .801        -.37       -.2
Resisting side:
  Effective earth loads......                 -.046         .67       -.0
  Water loads................                 -.131         .67       -.0
Foundation:
  Components of normal.......   -51.613      -7.579       -6.98     363.9
  Components of shear........     1.143      -7.781         .00        .0
  Uplift.....................    -3.140                   -8.11      25.4
=====================================================================
** Statics Check **   SUMS =       .000        .000                    .0

Angle of base          =     8.35 degrees
Normal force on base =   52.166 kips
Shear force on base  =    7.864 kips
Max. available shear force =   36.527 kips

Base pressure at heel =   3.2333 ksf
Base pressure at toe  =   3.9852 ksf
```

```
Xr (measured from toe) =     6.98 ft
Resultant ratio        =    .4826
Stem ratio             =    .0000
Base in compression    =   100.00 %
Overturning ratio      =     3.56

Volume of concrete =    12.72 cubic yds/ft of wall

NOTE:  The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures
do not result in excessive differential settlement of
the wall foundation.
```

```
************************
**  Sliding Results   **
************************

Solution converged.  Summation of forces = 0.

           Horizontal  Vertical
  Wedge      Loads      Loads
  Number    (kips)      (kips)
  ==================================
     1        .000        .000
     2        .000        .000
     3        .000        .000

  Water pressures on wedges:

             Top      Bottom
    Wedge    press.   press.   x-coord.    press.
    number   (ksf)    (ksf)     (ft)       (ksf)
  ================================================
      1     .0000     .3080
      2                           .0000      .3080
      2                        14.3000       .1311
      3     .0000     .1311

Points of sliding plane:
  Point 1 (left),  x =      .00 ft,   y =    16.80 ft
  Point 2 (right), x =    14.30 ft,   y =    18.90 ft

Depth of cracking =      .00 ft

          Failure   Total    Weight    Submerged   Uplift
  Wedge    angle    length  of wedge    length     force
  number   (deg)     (ft)    (kips)      (ft)      (kips)
  ========================================================
     1   -41.382    44.926     57.643    7.866     1.211
     2     8.354    14.453     43.291   14.453     3.174
     3    36.481     3.364       .325    3.364      .221

  Wedge     Net force
  number     (kips)
  ===================
     1      -19.991
     2       19.660
     3         .331
  ===================
     SUM =     .000

+-----------------------------+
| Factor of safety =    2.262 |
+-----------------------------+
```

DESIGN CHARACTERISTICS

DESIGN CHARACTERISTICS

1.0 Setting

The local study area is situated in an urban area of southwestern San Jose, in the extreme southern area of the highly urbanized Santa Clara Valley. Santa Clara Valley lies in the center of Santa Clara County in west central California, immediately south of San Francisco Bay. The county is bounded on the northeast by Alameda County, on the northwest by San Mateo County and the southwest by Santa Cruz County.

For descriptive purposes, the study area has been divided into "river reaches" and "economic areas" corresponding to the major bridge crossings and floodplain areas, respectively. River reaches are defined in Table 1 and shown in Figure 1 of the Hydrology and Hydraulics (H&H) Analysis section, and economic areas are shown in Plate 7.

Table 1: Upper Guadalupe Study River Reaches

River Reach Number	Approximate Project Station	From	To
7	740 - 781	SPRR river crossing	Union Pacific Railroad (UPRR) river crossing
8	781 - 797	UPRR river crossing	Willow Glen Way
9	797 - 843	Willow Glen Way	Curtner Avenue
10	843 - 909	Curtner Avenue	Capitol Expressway
11	909 - 940	Capitol Expressway	Branham Lane
12	940 - 1016	Branham Lane	Blossom Hill Road

2.0 Plans of Improvement - Recommended Plan

Three plans of improvement were evaluated: a 20-year plan, 50-year plan and 100-year plan. These plans feature floodwalls, channel widening, a channel bypass and bridge modifications/replacement as means to alter the upper Guadalupe River from Highway 280 upstream to Blossom Hill Road, a distance of about 5.5 miles. Improvements on Ross Creek will be constructed from Guadalupe River to Jarvis Avenue. For all designs (20-, 50- and 100-year) it is assumed that a project with a 20-year level of protection will be constructed on Canoas Creek.

2.1 Bypass Channel Plan Designed by the Local Sponsor

The 100-year plan features a bypass channel, channel widening, levee and floodwalls designed to contain the 100-year discharge on the upper Guadalupe River and Ross Creek. Channel widening will be limited to one bank in most cases to preserve the existing riparian habitat. A plan view of the 100-Year Plan is shown on Plate 10. The plan is described below.

Highway 280 to 500' upstream of Willow Street: Improvements to this project section will include an 18'-deep bypass channel with an 85'-wide floodway and 1:1 side slopes.

500' upstream of Willow Street to Alma Street: Improvements to this project section will consist of a combined natural and bypass channel. 18'-deep bypass with 30'-wide floodway and 1:1 side slopes; the surface elevation of the embankment between the bypass and natural channel would allow transfer of floodwaters between the two alignments during high flows.

Alma Street to UPRR Bridge: A gabion lined bypass channel will be built through the existing Elks Lodge parking lot.

UPRR Bridge to Willow Glen Way: 18'-deep bypass with 85'-wide channel floodway with 1:1 side slopes.

Willow Glen Way to Blossom Hill Road: Improvements to this project section will include channel widening. The bank to bank width of the project will range from 75 feet to 200 feet, 4-10 feet above the invert (See Figure 2 of the Hydrology and Hydraulics Appendix).

2.1.1 Canoas Creek

Canoas Creek will be improved to provide a 20-year level of protection.

2.1.2 Ross Creek

Improvements to Ross Creek will include new culverts and channel widening. The channel bottom width be excavated to 35 feet from Almaden Expressway to a point 750 feet upstream of Jarvis Avenue. The existing 12'W x 10'H x 210'-long box culvert at Almaden Expressway will be replaced with a 20'W x 10'H x 210'-long culvert. At Jarvis Avenue two12'W x 9.5'H culverts will be installed in addition to the existing 12'W x 9.5'H culvert.

2.2. Structural Requirements

2.2.1. Design Criteria

Structural design of the flood walls and bridges will be in accordance with applicable Corps of Engineers Engineering Manuals and Regulations, supplemented where necessary by applicable industry codes and recognized design standards. The bridge approach retaining walls will be designed in accordance with the CalTrans bridge design manual.

2.2.2. Reference

The following are the principal references used in the structural stability analysis and design.

EM 1110-2-2502	Retaining and Flood Walls
EM 1110-2-2104	Strength Design for Reinforced Concrete Hydraulic Structures
EM 1110-2-2000	Standard Practice for Concrete
EM 1110-2-2102	Waterstops and Other Joint Materials

American Association of State Highway and Transportation Officials Interim Specifications-Bridges

USS Steel Sheet Piling Design Manual

CalTrans Bridge Design Specifications Manual

2.2.3. Materials

Concrete	f'c= 3000psi @ 28 days
Prestress Concrete	f'c= 5000psi @ 28 days
Reinforcing Steel	ASTM A 615 Grade 60
Concrete Masonry Units	ASTM C90 Grade N
Steel Sheet Piles	ASTM A328

2.2.4. Design Data and Assumptions

Soil Properties

Friction Angle (phi) = 30
Allowable Bearing Capacity = 3000psf
Unit Moist Weight = 125pcf
Submerged Weight = 63pcf
Earthquake Load on Walls = 0.20g

Wind Load	22 psf on vertical projection
Bridge Loading	AASHTO HS-20
Earthquake Load on Bridge	CalTrans Spectra per Reference 7

Stability Criteria for Flood and Retaining Walls (Minimum Base Area in Compression)

	Sliding	Overturning	Bearing
Design Flood	1.5	100%	3
Water to top of wall	1.33	75%	2
Earthquake/Wind	1.1	Resultant within base	> 1

3.0 Relocation & Replacement Requirements

Table 2 lists the relocation and replacement requirements by reach for roads, bridges, and utilities:

Table 2: Relocations & Replacements

Reach	Type	Approximate location
7A	Bridge removal & replacement	Willow Street
	Utility relocation - sanitary sewer	Willow Street
	Roadway replacement	Willow Street
7B	Utility relocation - water lines	Alma Ave. bridge
	Roadway replacement	Alma Ave. & Elk's Lodge
8	Utility relocation - outfalls	Bypass channel
	Roadway replacement	Bypass channel
9	Bridge removal & replacement	Willow Glen Way
	Utility relocation - pumps, etc.	Willow Glen Way
10A	Bridge removal & replacement	Curtner Avenue
	Utility relocation - outfalls	Curtner Avenue
	Roadway replacement	Curtner Avenue
10C & D	Bridge removal & replacement	Hillsdale Avenue
	Utility relocation - sanitary sewer	Sta. 889+20
11	Utility relocation/mod. - outfalls	Bryan Ave. Station
Canoas Creek	Roadway replacement	Almaden Expwy.
Ross Creek	Utility relocation - outfalls	N. bank only
	Roadway replacement	

CONSTRUCTION SCHEDULE

Schedule for Design and Construction

The Feasibility Study completion is scheduled for January, 1998 with the issuance of the Division Engineer's Public Notice. Plans and Specifications (P&S) will be performed by the San Francisco District Corps of Engineers. Completion of P&S is scheduled at the end of September 1999.

Due to budget ceilings, funding for construction is not scheduled for availability before 2007. However, the following schedule is based on the premise that funds will become available in a more timely manner, and thus, assumes the most optimistic schedule possible. The Corps will officially request the Sponsor to acquire the necessary real estate (LERRD) with the signing of the PCA in March 2000. The final certification of real estate would follow one year later in March 2001. The advertisement of the construction contract is scheduled in June 2001. Construction is scheduled to commence in August 2001 with a construction performance period of three years. Final acceptance and transfer to the Sponsor is scheduled at the end of May 2005, following the delivery of an O&M Manual and As-Built Drawings to the Sponsor.

The estimated schedule for completion of work items is as follows:

ITEM	COMPLETION DATE
9.5.1 Division Engineer's Public Notice	Mar 98
9.5.2 Initiate P&S	May 98
9.5.3 Plans and Specifications	May 00
9.5.4 Approval New Construction Start	Dec 98
9.5.5 PCA Signed	30 Mar 00
9.5.6 Real Estate Acquisitions Complete	30 Mar 01
9.5.7 Advertise Construction Contract	30 Jun 01
9.5.8 Award of Construction Contract	15 Aug 01
9.5.9 Completion of Construction	30 Sep 04

PLATES

1. Extent of January 1995 Flooding
2. Extent of March 1995 Flooding
3. 20-year Floodplain
4. 50-year Floodplain
5. 100-year Floodplain
6. 500-year Floodplain
7. Economic Areas
8. Major Features of Willow Glen Plan
9. Major Features of Valley View Plan
10. Major Features of Bypass Channel Plan
11. Residual Floodplain (20-year)
12. Residual Floodplain (50-year)
13. Residual Floodplain (100-year)
14. Geologic Surface Map
15. Fault Map
16. Project Location
17. Soil Classification
18. Boring Logs
19. Boring Logs
20. Boring Logs
21. Boring Logs

DESCRIPTION OF MAP UNITS

GP	Gravel Pits-- Excavations in stream channels and Holocene alluvium.
Qhsc	Stream Channels and Their Deposits (Holocene)-- Poorly- to well -sorted sandy silt, silty sand, sand or sandy gravel with minor cobbles.
Qhfp1	Alluvial Terrace Deposits (Holocene)--Qhfp1, cut into levee (Qhl) deposits. Deposits are generally less than 1 meter thick and consist of rounded gravel and historic artifacts in a clayey silt matrix.
Qhl	Natural Levee Deposits (Holocene)--Loose, moderate- to well-sorted sandy or clayey silt grading to sandy or silty clay.
Qhfp	Floodplain Deposits (Holocene)--Medium to dark gray, dense, sandy to silty clay. Lenses of coarser material (silt, sand, and pebbles) may be locally present.
Qhb	Floodbasin Deposits (Holocene)--Organic-rich dark clay to very fine silty-clay deposits occupying the lowest topographic positions between either the Holocene levee deposits (Qhl) or Holocene floodplain deposits (Qhfp).
Qhaf	Alluvial Fan Deposits (Holocene)--Brown medium dense, gravelly sand and clayey gravel that grade upward to sandy or silty clay. Sediments near the heads of these fans are typically brown or tan, medium dense to dense, gravelly sand or sandy gravel that grades upward to sandy or silty clay.
Qpaf	Alluvial Fan Deposits (Pleistocene)--Tan to reddish brown, dense, gravelly and clayey sand or clayey gravel that grades upward to a sandy clay.

Undifferentiated Bedrock--Rocks older than Pliocene-Pleistocene have not been differentiated.

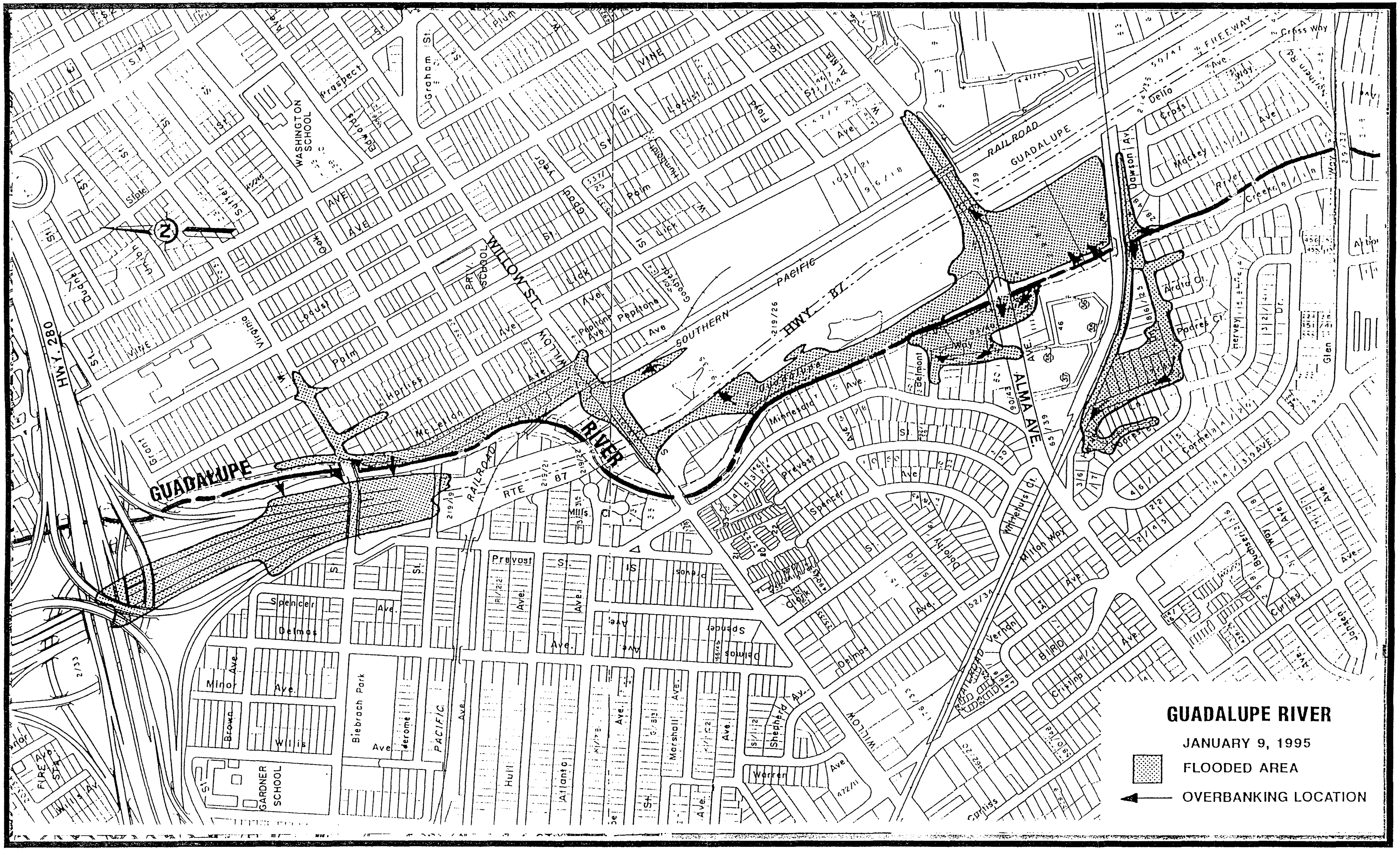
SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
FEASIBILITY STUDY

MAP UNITS

IN SHEET SHEET NO
U S ARMY ENGINEER DIST. SAN FRANCISCO. C OF E
DRAWN FILE NO
TRACED TO ACCOMPANY REPORT
CHECKED DATED

PLATE 14b



GUADALUPE RIVER

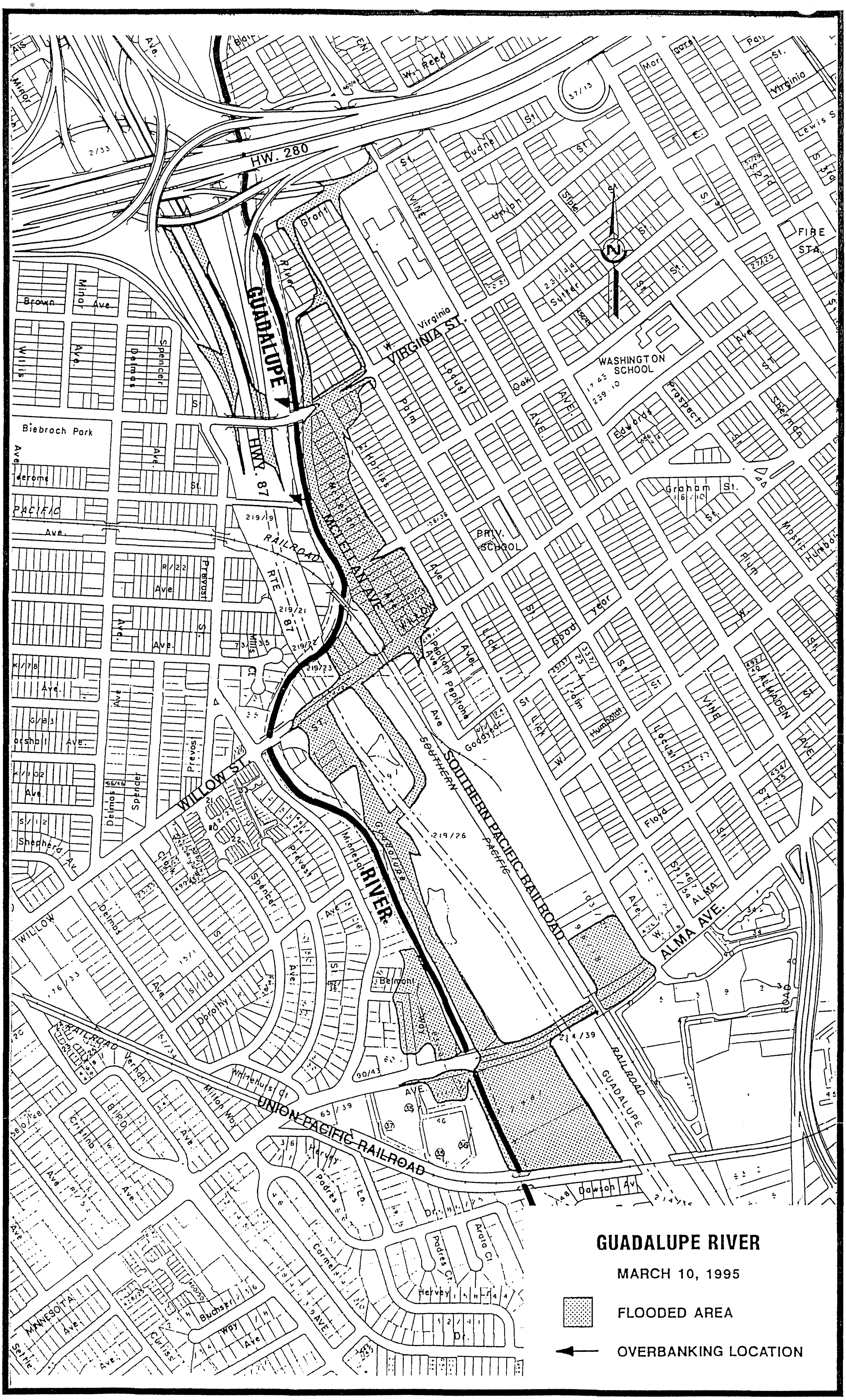
JANUARY 9, 1995

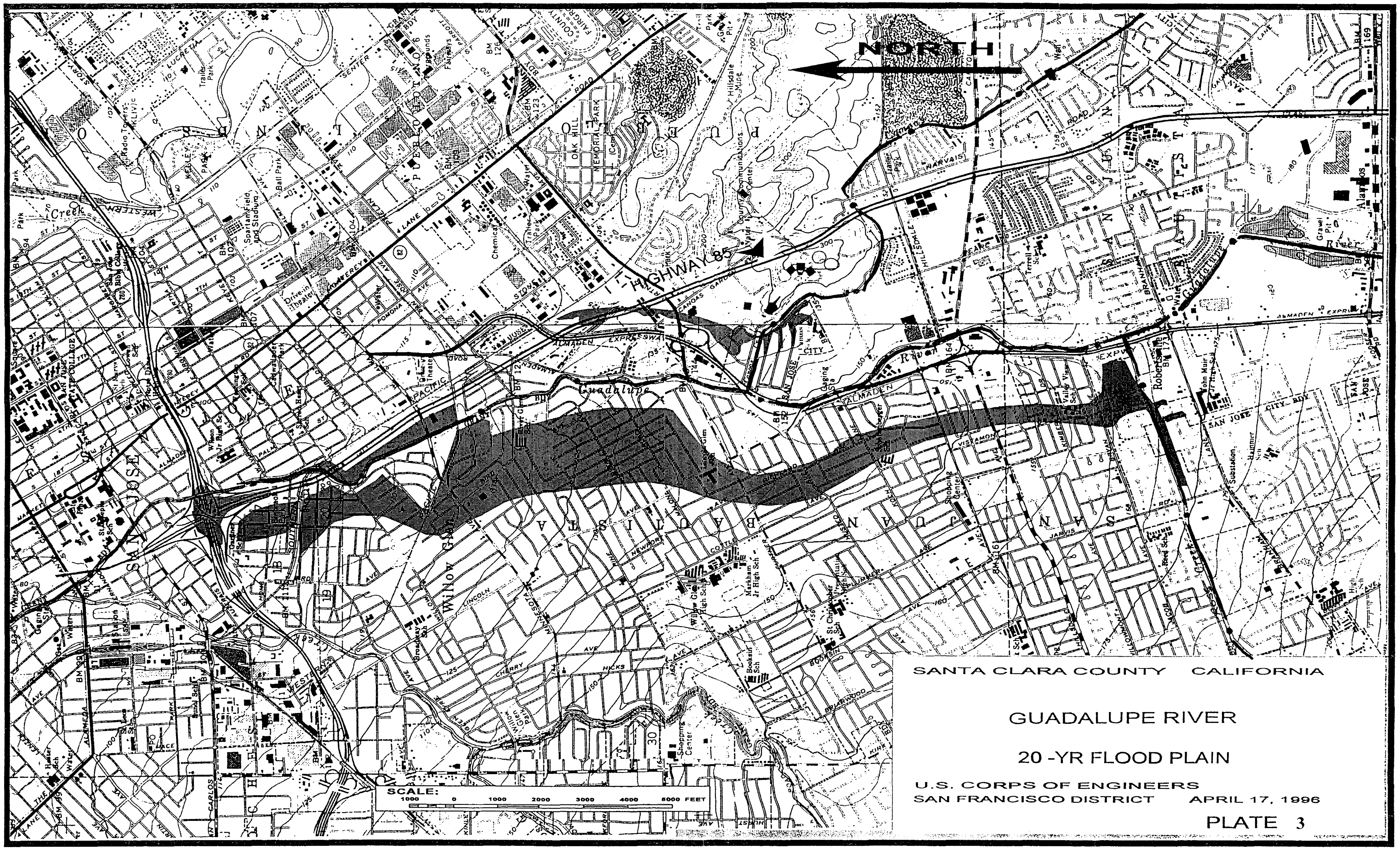
▨ FLOODED AREA

← OVERBANKING LOCATION

PLATE 1

PLATE 2


GUADALUPE RIVER
MARCH 10, 1995
FLOODED AREA
OVERBANKING LOCATION
HW. 280
HW. 87
RTE 87
GUADALUPE
RIVER
McLELLAN AVE
VIRGINIA ST.
WASHINGTON SCHOOL
PRIV. SCHOOL
FIRE STA.
Biebrach Park
WILLOW ST.
WILLOW
SOUTHERN PACIFIC RAILROAD
UNION PACIFIC RAILROAD
RAILROAD
ALMA AVE.
ALMADEN AVE
Graham St.
Edwards
Prospect
Goodyear
Lick
Locust
Palm
Vine
Oak
Delmas
Spencer
Prevost
Minor
Brown Ave
Willis
Minnesota Ave.
Carmel
Dawson Av.
Hervey Ln.
Padres Ct.
Aralia Ct.
Belmont
Dorothy
Floyd
Humboldt
Sutter
Union
Dupont
Grant



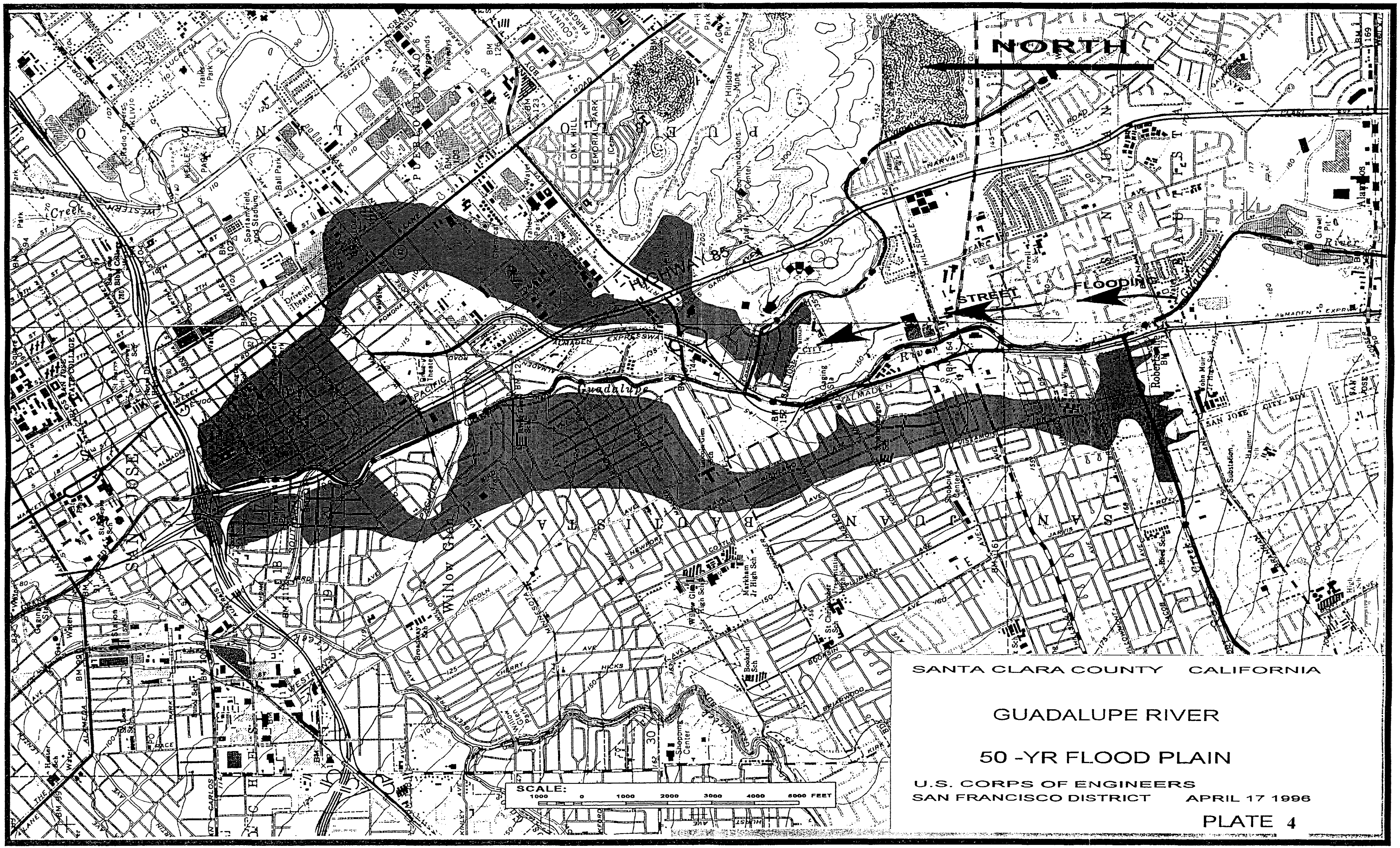
SANTA CLARA COUNTY CALIFORNIA

GUADALUPE RIVER

20 -YR FLOOD PLAIN

U.S. CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT APRIL 17, 1996

PLATE 3



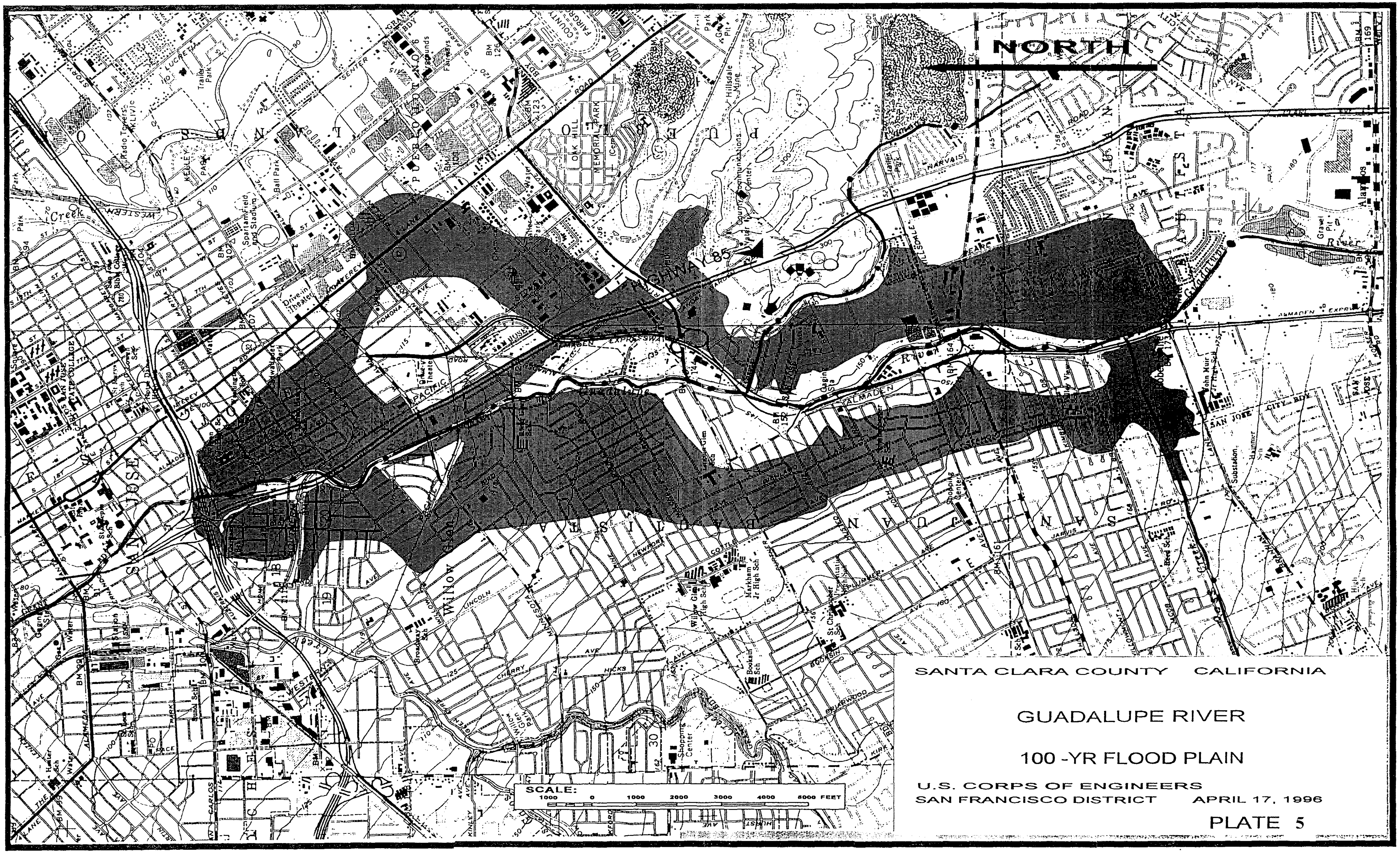
SANTA CLARA COUNTY CALIFORNIA

GUADALUPE RIVER

50 -YR FLOOD PLAIN

U.S. CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT APRIL 17 1996

PLATE 4



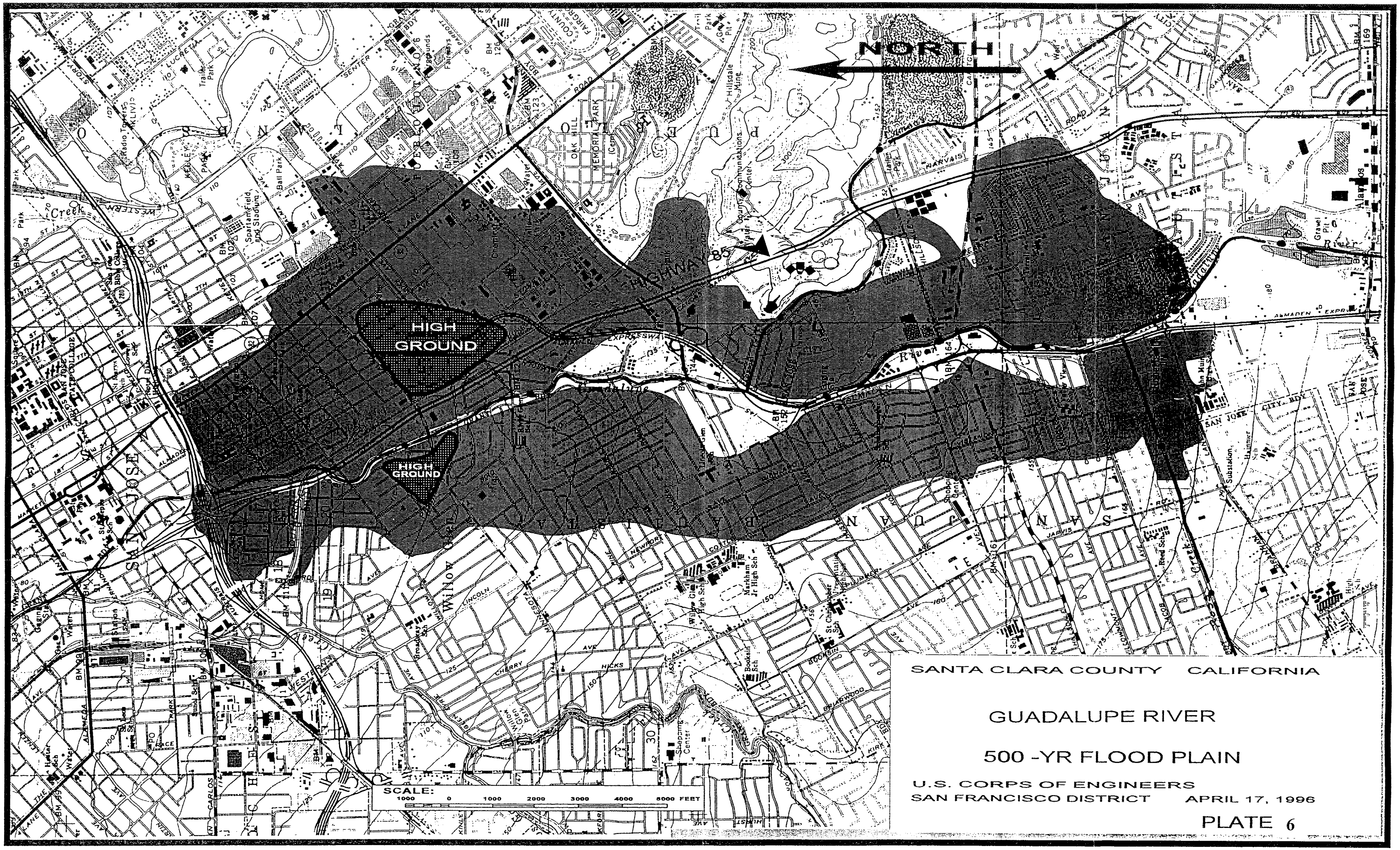
SANTA CLARA COUNTY CALIFORNIA

GUADALUPE RIVER

100 -YR FLOOD PLAIN

U.S. CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT APRIL 17, 1996

PLATE 5



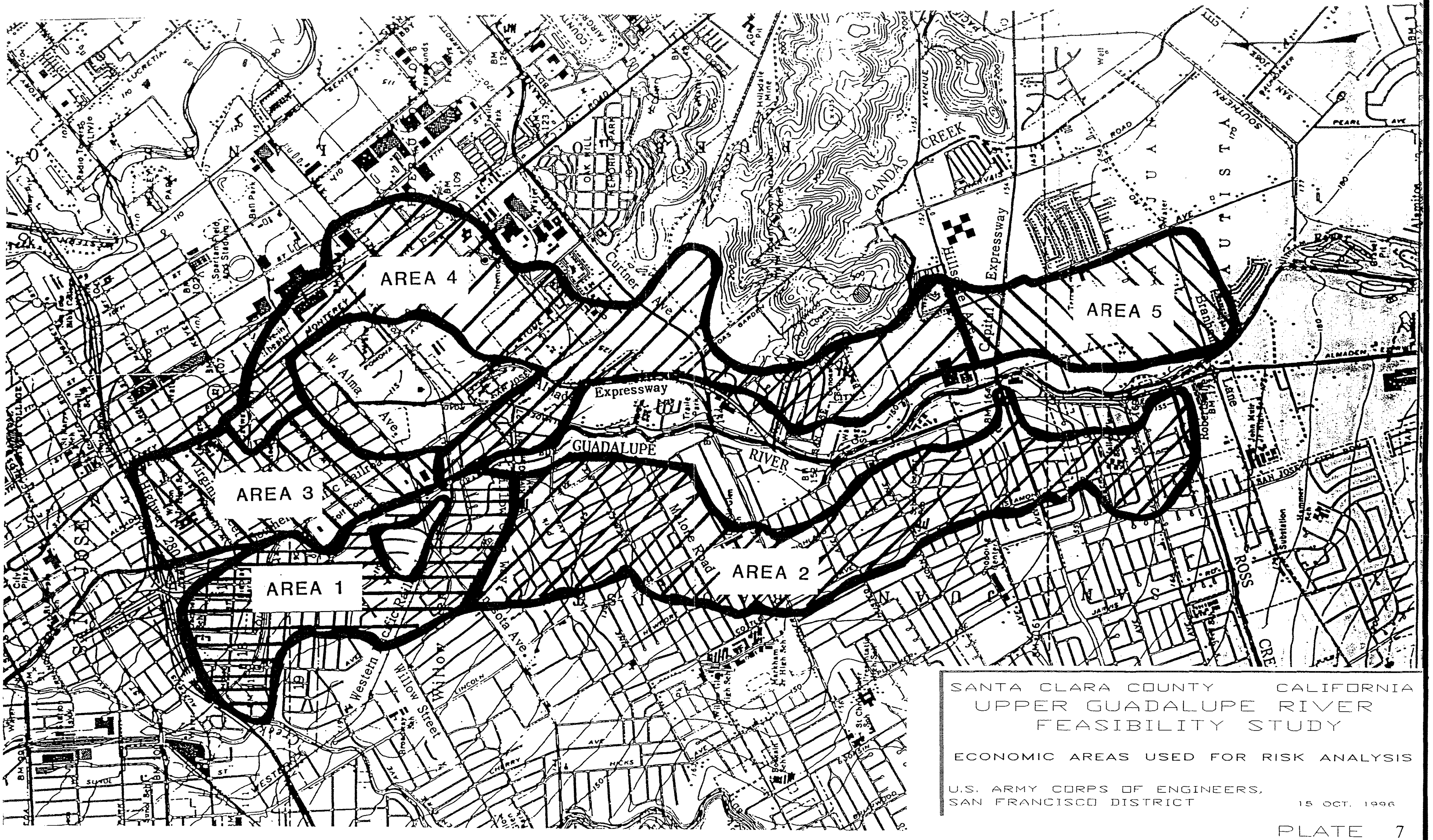
SANTA CLARA COUNTY CALIFORNIA

GUADALUPE RIVER

500 -YR FLOOD PLAIN

U.S. CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT APRIL 17, 1996

PLATE 6

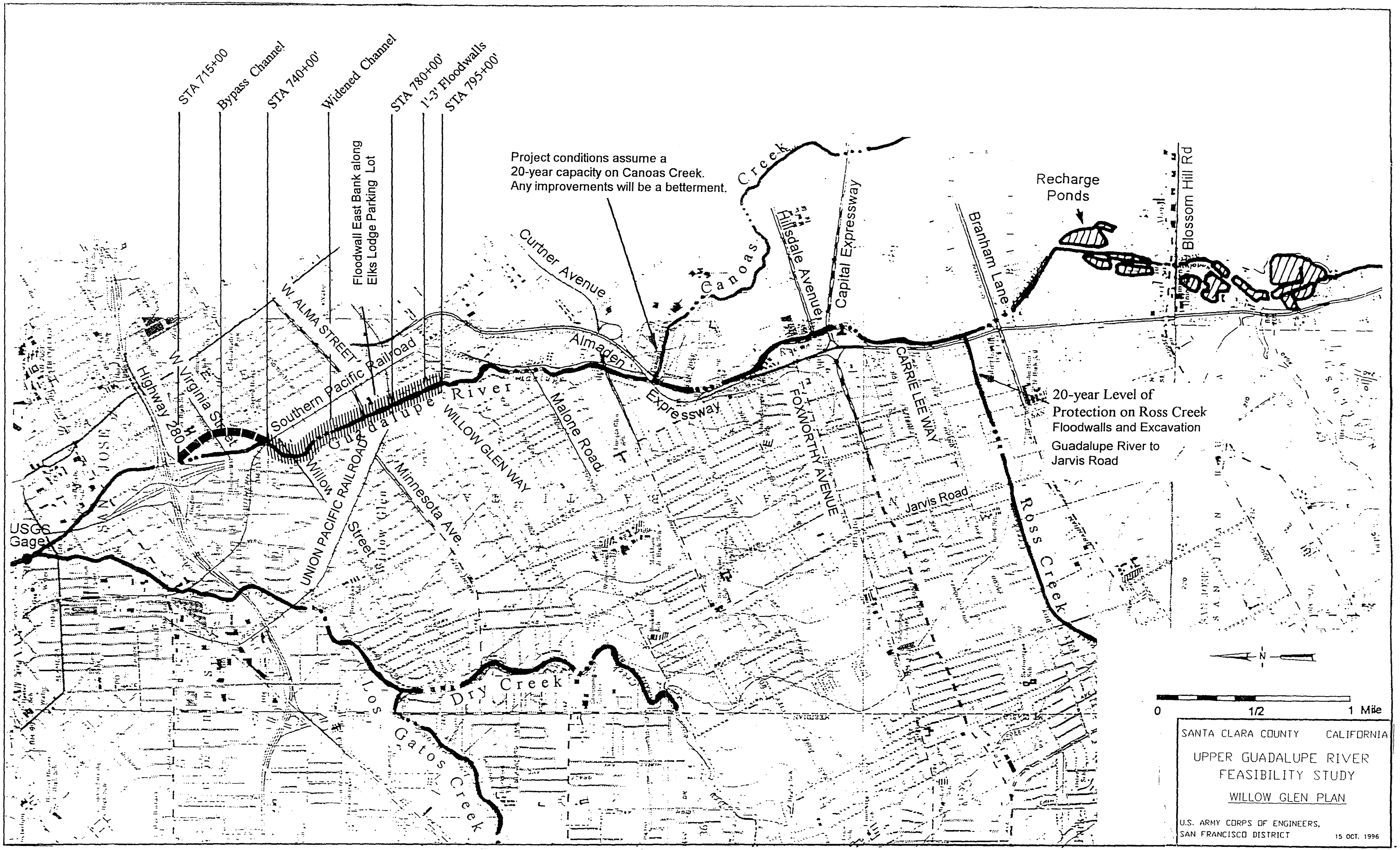


SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FEASIBILITY STUDY

ECONOMIC AREAS USED FOR RISK ANALYSIS

U.S. ARMY CORPS OF ENGINEERS,
SAN FRANCISCO DISTRICT 15 OCT. 1996

PLATE 7



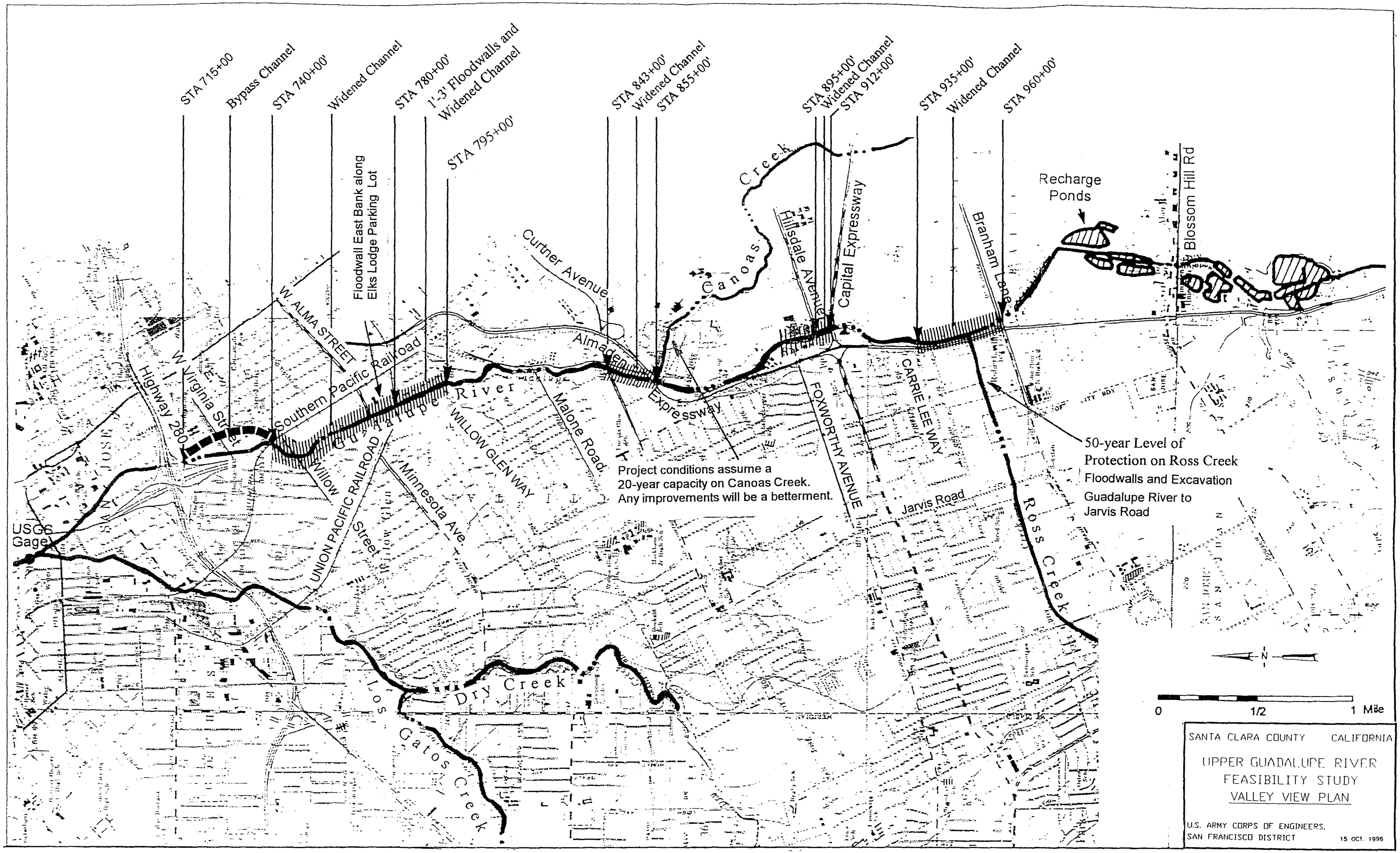
SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
FEASIBILITY STUDY

WILLOW GLEN PLAN

U.S. ARMY CORPS OF ENGINEERS,
SAN FRANCISCO DISTRICT 15 OCT. 1996

PLATE 8



Project conditions assume a 20-year capacity on Canoas Creek. Any improvements will be a betterment.

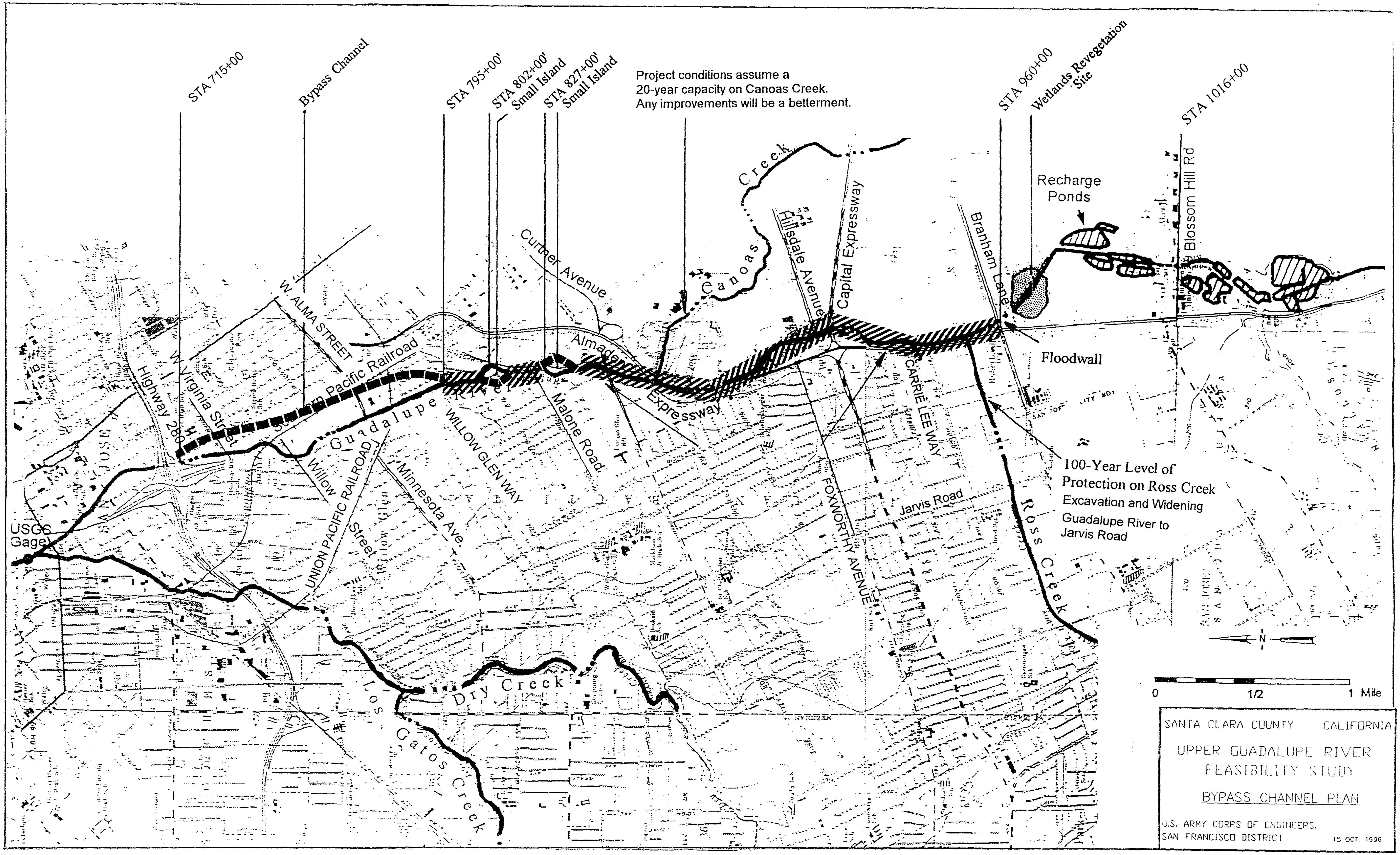
50-year Level of Protection on Ross Creek
Floodwalls and Excavation
Guadalupe River to Jarvis Road

SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
FEASIBILITY STUDY
VALLEY VIEW PLAN

U.S. ARMY CORPS OF ENGINEERS,
SAN FRANCISCO DISTRICT 15 OCT. 1996

PLATE 9



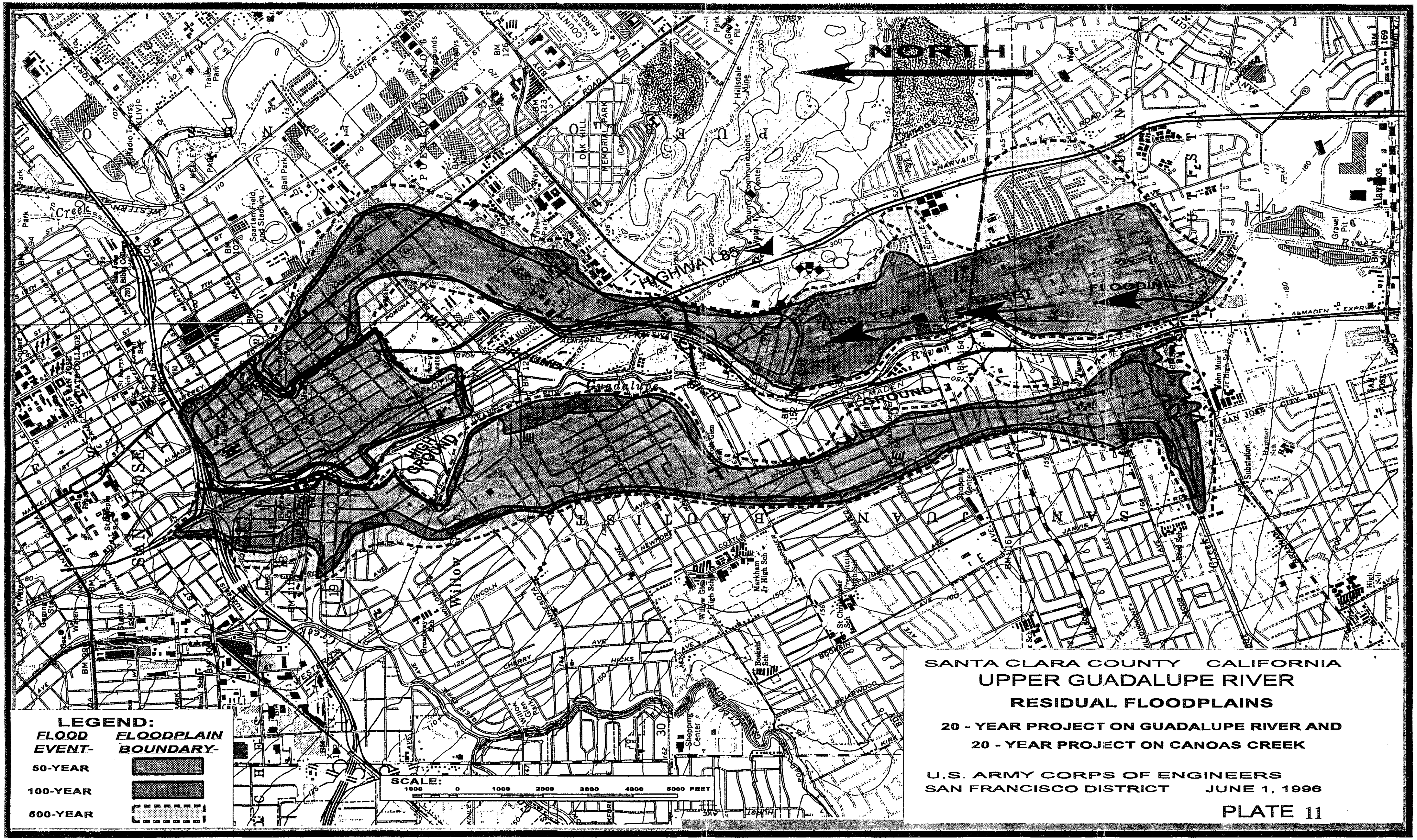
SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
FEASIBILITY STUDY

BYPASS CHANNEL PLAN

U.S. ARMY CORPS OF ENGINEERS,
SAN FRANCISCO DISTRICT 15 OCT. 1996

PLATE 10



SANTA CLARA COUNTY CALIFORNIA

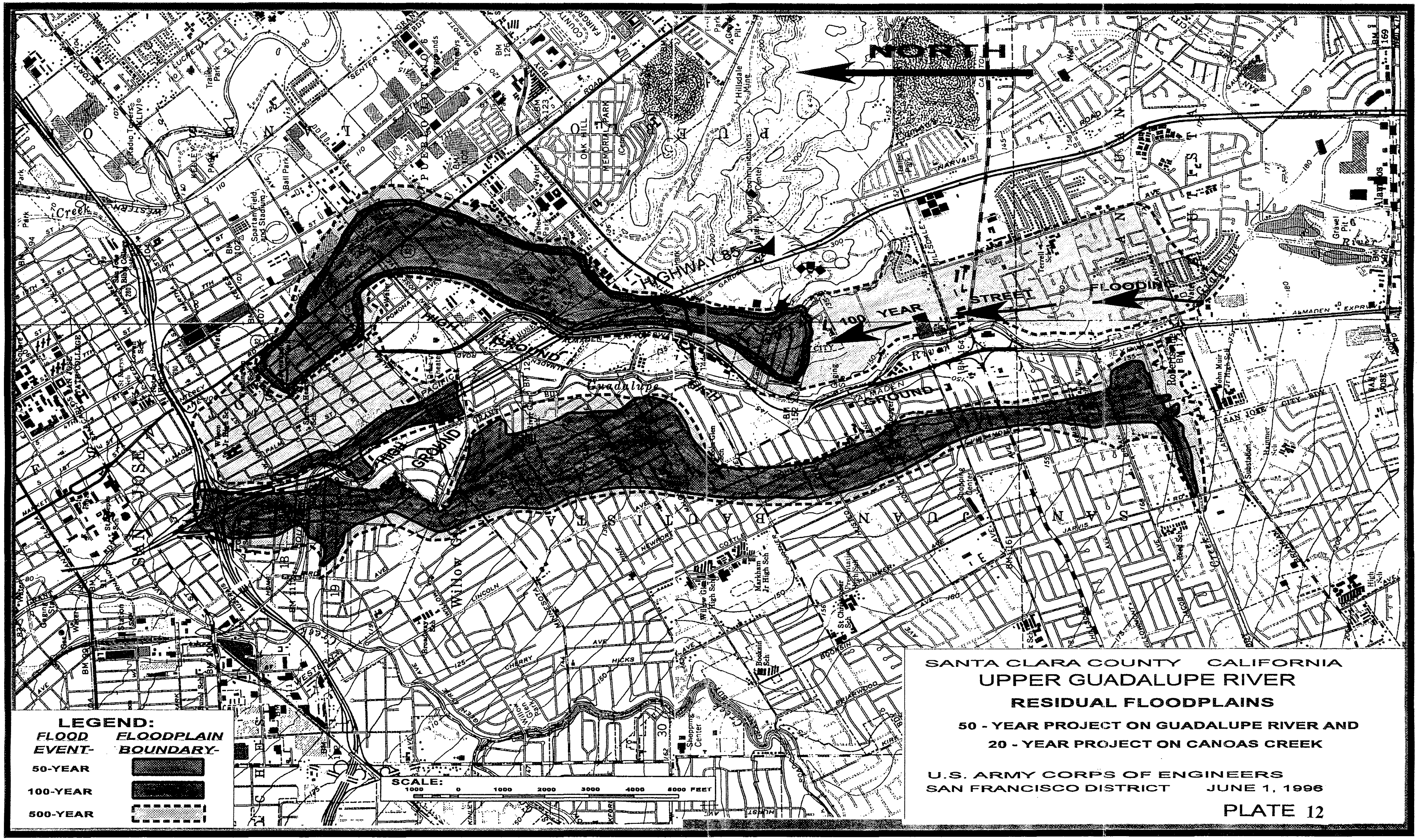
UPPER GUADALUPE RIVER

RESIDUAL FLOODPLAINS

20 - YEAR PROJECT ON GUADALUPE RIVER AND
20 - YEAR PROJECT ON CANOAS CREEK

U.S. ARMY CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT JUNE 1, 1996

PLATE 11



LEGEND:

FLOOD EVENT-	*FLOODPLAIN BOUNDARY-*
50-YEAR	▨
100-YEAR	■
500-YEAR	⬚

SANTA CLARA COUNTY CALIFORNIA

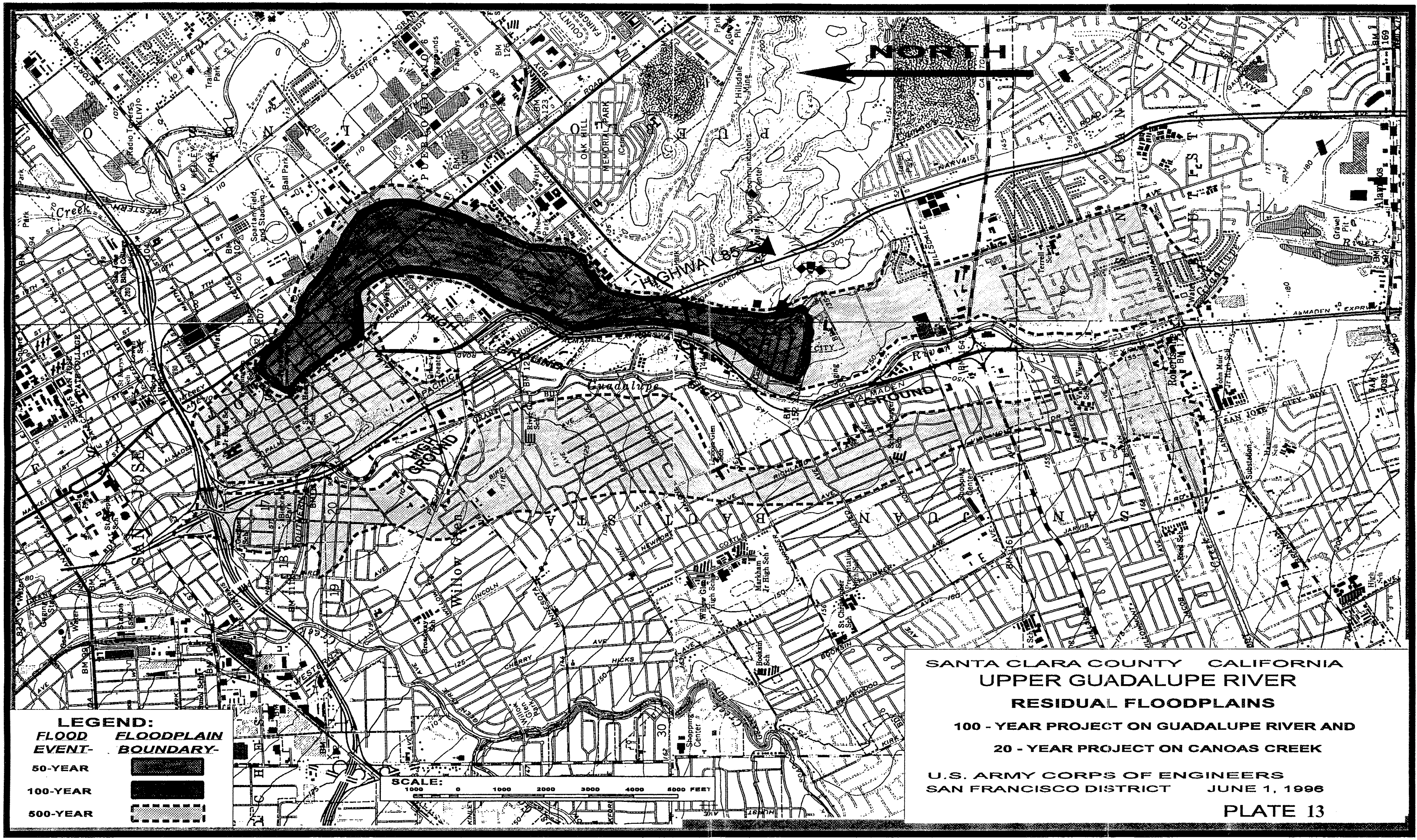
UPPER GUADALUPE RIVER

RESIDUAL FLOODPLAINS

50 - YEAR PROJECT ON GUADALUPE RIVER AND
20 - YEAR PROJECT ON CANOAS CREEK

U.S. ARMY CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT JUNE 1, 1996

PLATE 12



SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER

RESIDUAL FLOODPLAINS

100 - YEAR PROJECT ON GUADALUPE RIVER AND

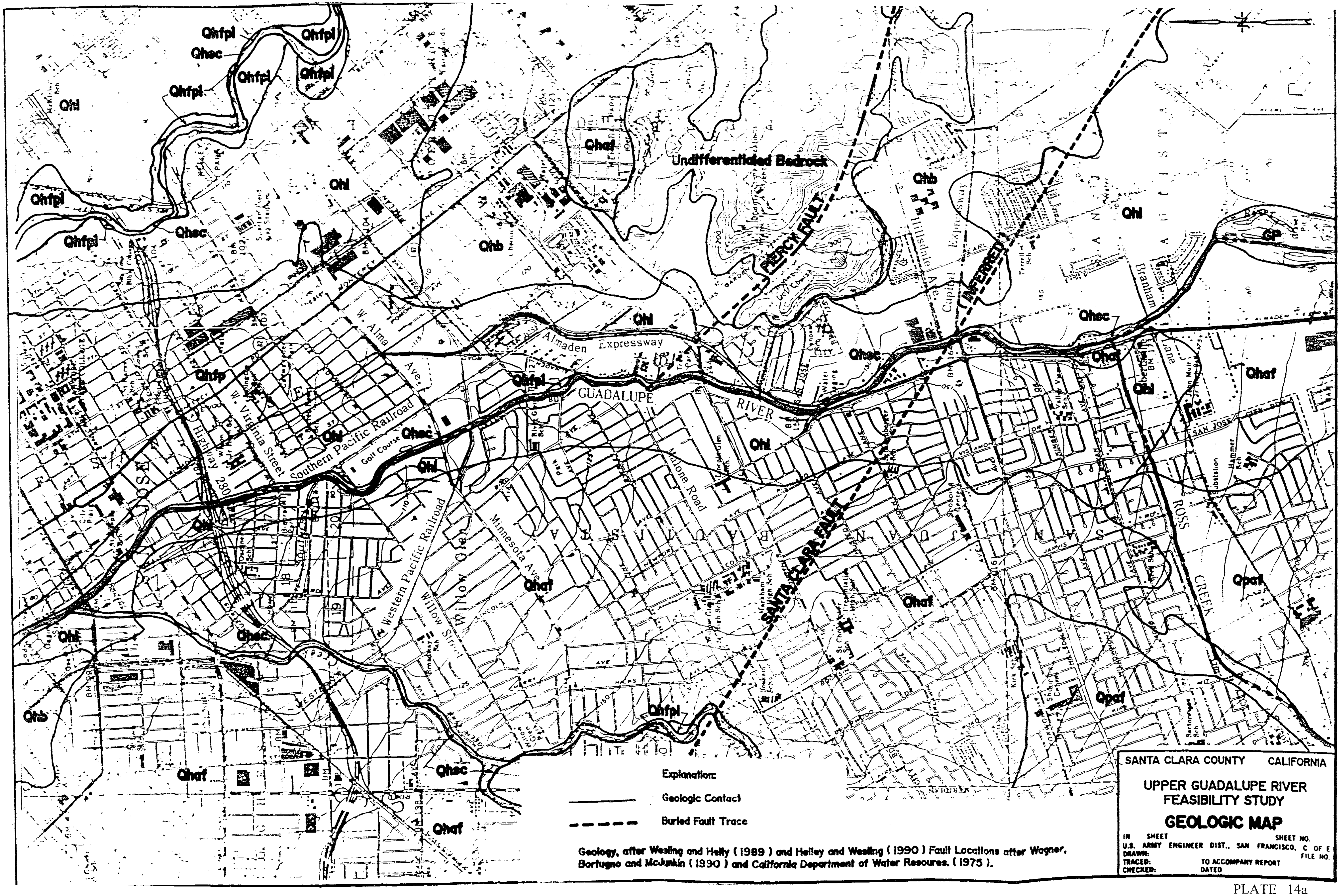
20 - YEAR PROJECT ON CANOAS CREEK

U.S. ARMY CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT JUNE 1, 1996

PLATE 13

LEGEND:

FLOOD EVENT-	*FLOODPLAIN BOUNDARY-*
50-YEAR	▨
100-YEAR	■
500-YEAR	▒



Explanation:

Geologic Contact

Buried Fault Trace

Geology, after Wesling and Helly (1989) and Helley and Wesling (1990) Fault Locations after Wagner, Bortugno and McJunkin (1990) and California Department of Water Resoures. (1975).

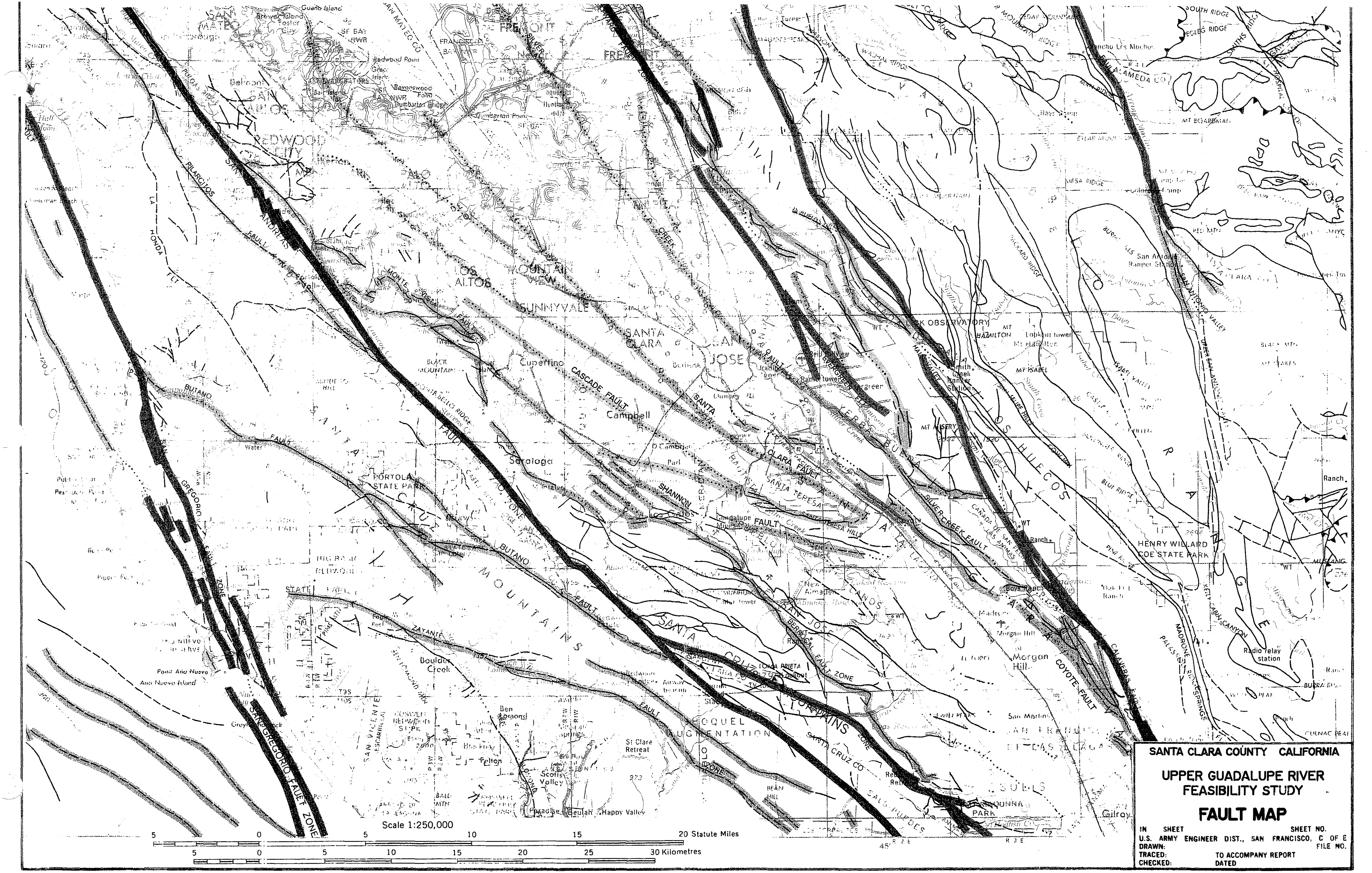
SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
FEASIBILITY STUDY

GEOLOGIC MAP

IN SHEET SHEET NO.
U.S. ARMY ENGINEER DIST., SAN FRANCISCO, C OF E
DRAWN: FILE NO.
TRACED: TO ACCOMPANY REPORT
CHECKED: DATED

PLATE 14a



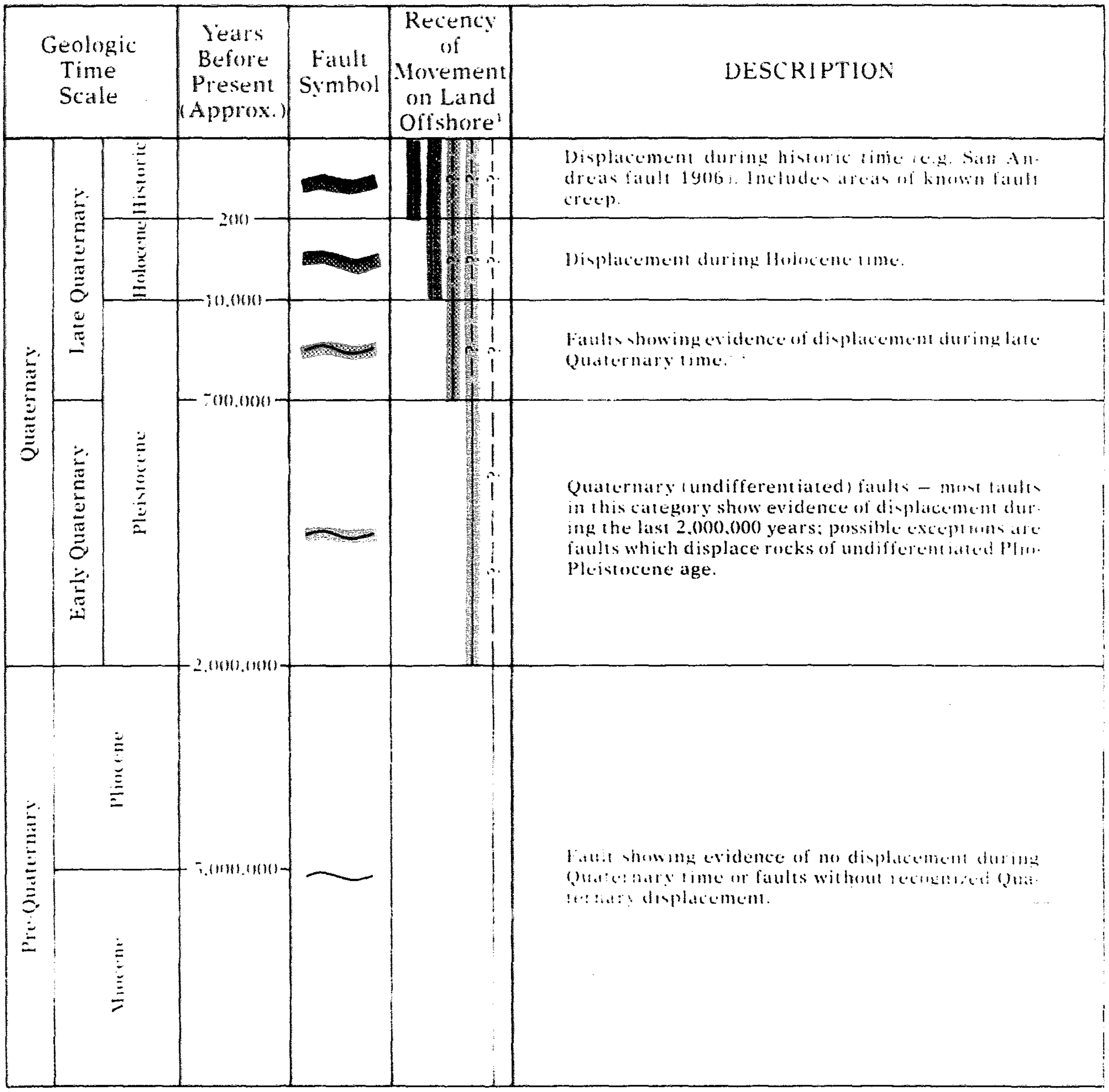
SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
FEASIBILITY STUDY

FAULT MAP

IN SHEET SHEET NO.
U.S. ARMY ENGINEER DIST., SAN FRANCISCO, C OF E
DRAWN: FILE NO.
TRACED: TO ACCOMPANY REPORT
CHECKED: DATED

PLATE 15a



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	200			Displacement during Holocene time.
		Pleistocene	10,000			Faults showing evidence of displacement during late Quaternary time.
	Early Quaternary		700,000			Quaternary (undifferentiated) faults – most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		2,000,000			
	Miocene		5,000,000			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

The fault map and the above Recency of Faulting Chart were reproduced directly from sheet 5 of 5 of Map No. 5A (Geology), San Francisco- San Jose Quadrangle; conpiled by D. L. Wagner, E. J. Bortugno, and R. D. McJunkin (1990), California Department of Conservation, Division of Mine, and Geology.

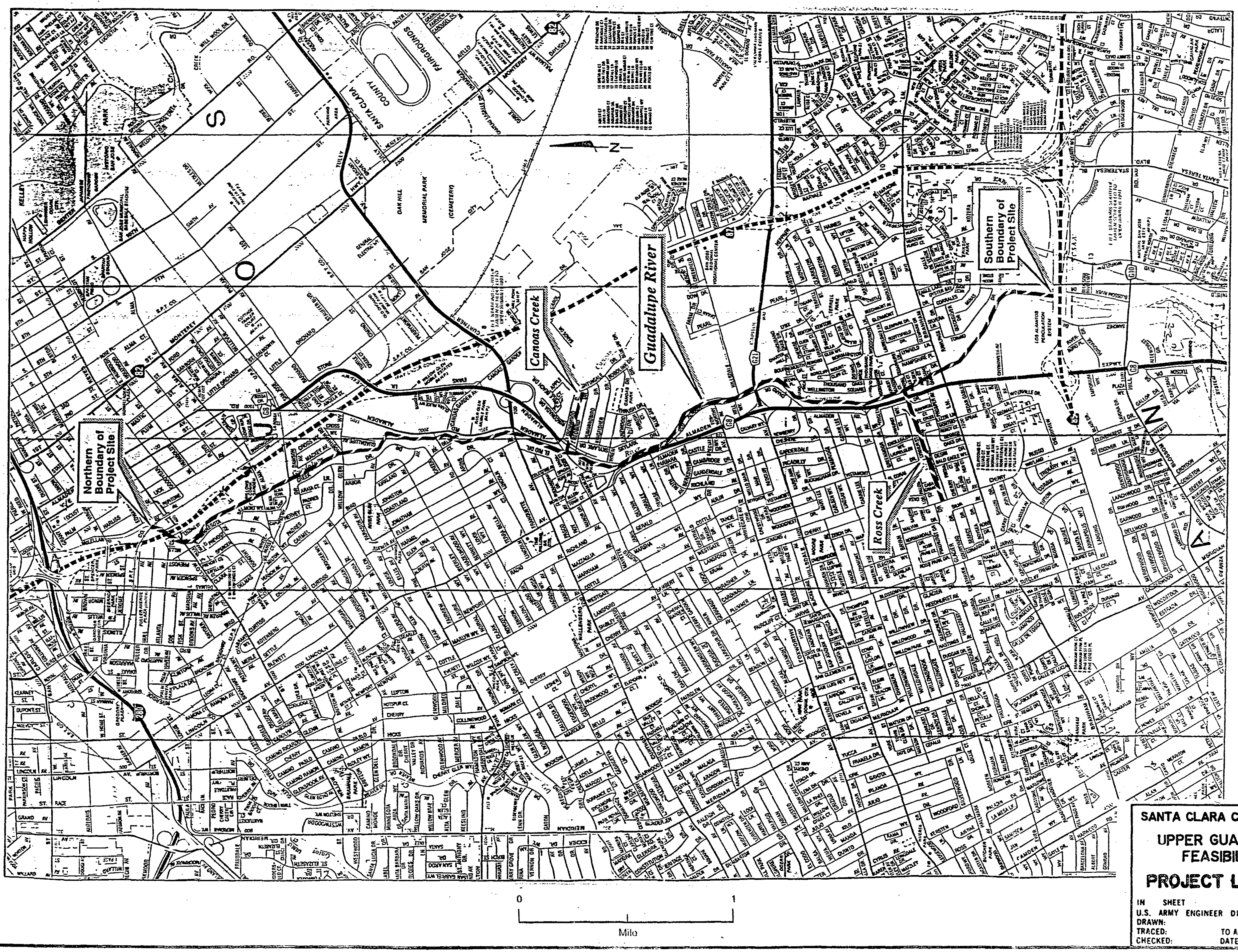
SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
FEASIBILTY STUDY

RECENCY OF FAULT

IN SHEET SHEET NO
U S ARMY ENGINEER DIST. SAN FRANCISCO, C OF E
DRAWN FILE NO
TRACED TO ACCOMPANY REPORT
CHECKED DATED

ENGINEERING DRAWINGS


Northern Boundary of Project Site
Southern Boundary of Project Site
Guadalupe River
Canoas Creek
Ross Creek
SANTA CLARA COUNTY FAIRGROUNDS
OAK HILL MEMORIAL PARK (CEMETERY)
0
1
Mile

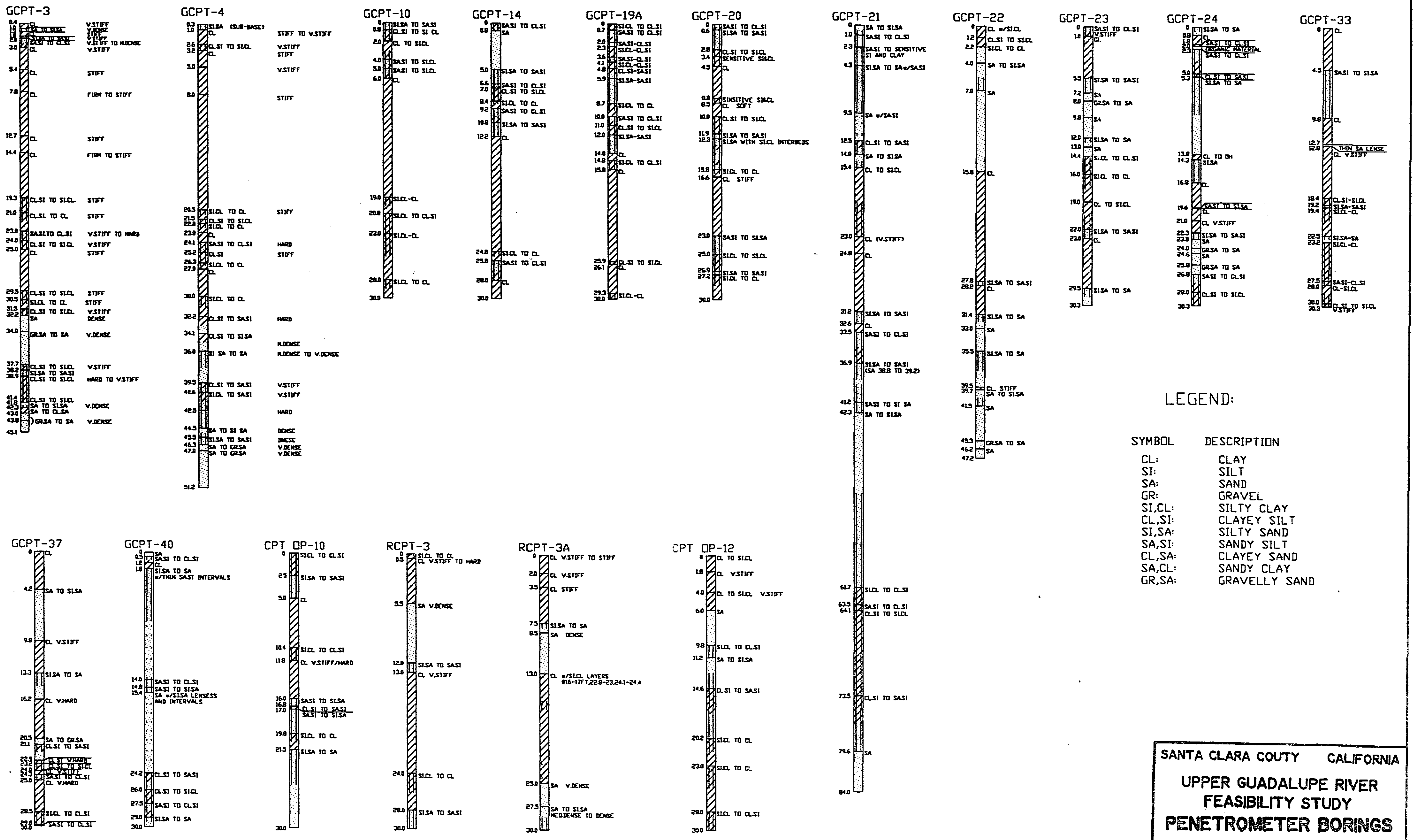
SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
FEASIBILITY STUDY

PROJECT LOCATION MAP

IN SHEET SHEET NO.
U.S. ARMY ENGINEER DIST., SAN FRANCISCO, C OF E
DRAWN: FILE NO.
TRACED: TO ACCOMPANY REPORT
CHECKED: DATED

PLATE 16



LEGEND:

SYMBOL	DESCRIPTION
CL:	CLAY
SI:	SILT
SA:	SAND
GR:	GRAVEL
SI,CL:	SILTY CLAY
CL,SI:	CLAYEY SILT
SI,SA:	SILTY SAND
SA,SI:	SANDY SILT
CL,SA:	CLAYEY SAND
SA,CL:	SANDY CLAY
GR,SA:	GRAVELLY SAND

SANTA CLARA COUTY CALIFORNIA

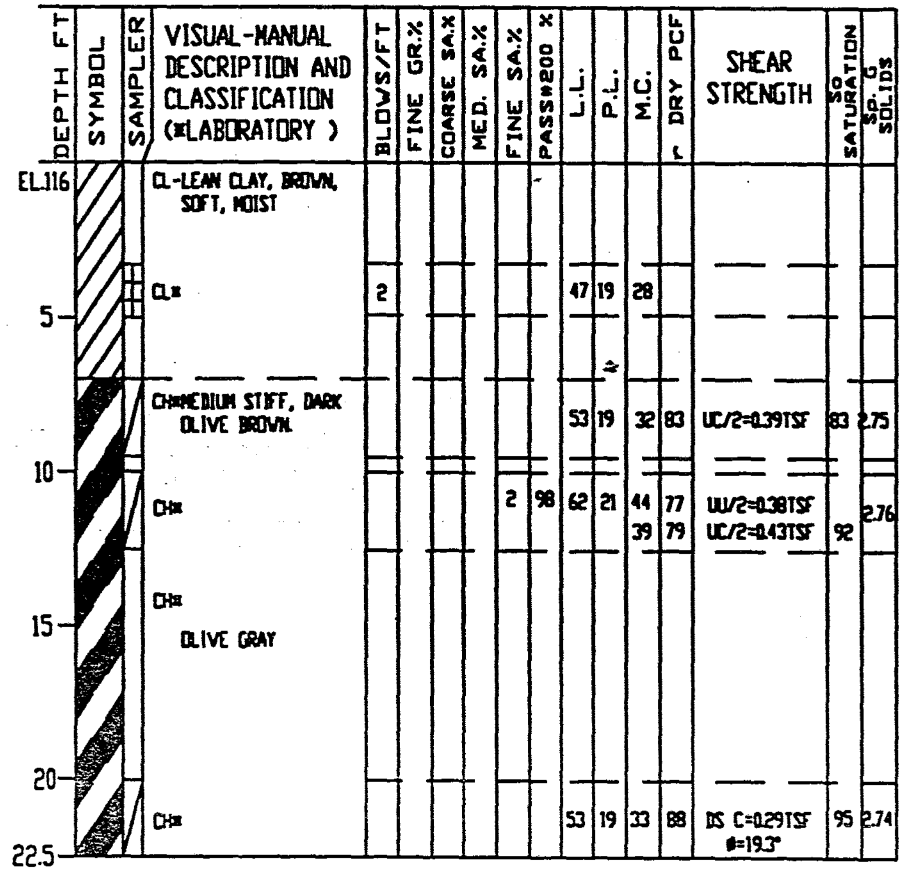
UPPER GUADALUPE RIVER FEASIBILITY STUDY

PENETROMETER BORINGS

IN SHEET
SHEET NO
U S ARMY ENGINEER DIST. SAN FRANCISCO, C OF E
DRAWN
FILE NO
TRACED
TO ACCOMPANY REPORT
CHECKED
DATED

PLATE 17

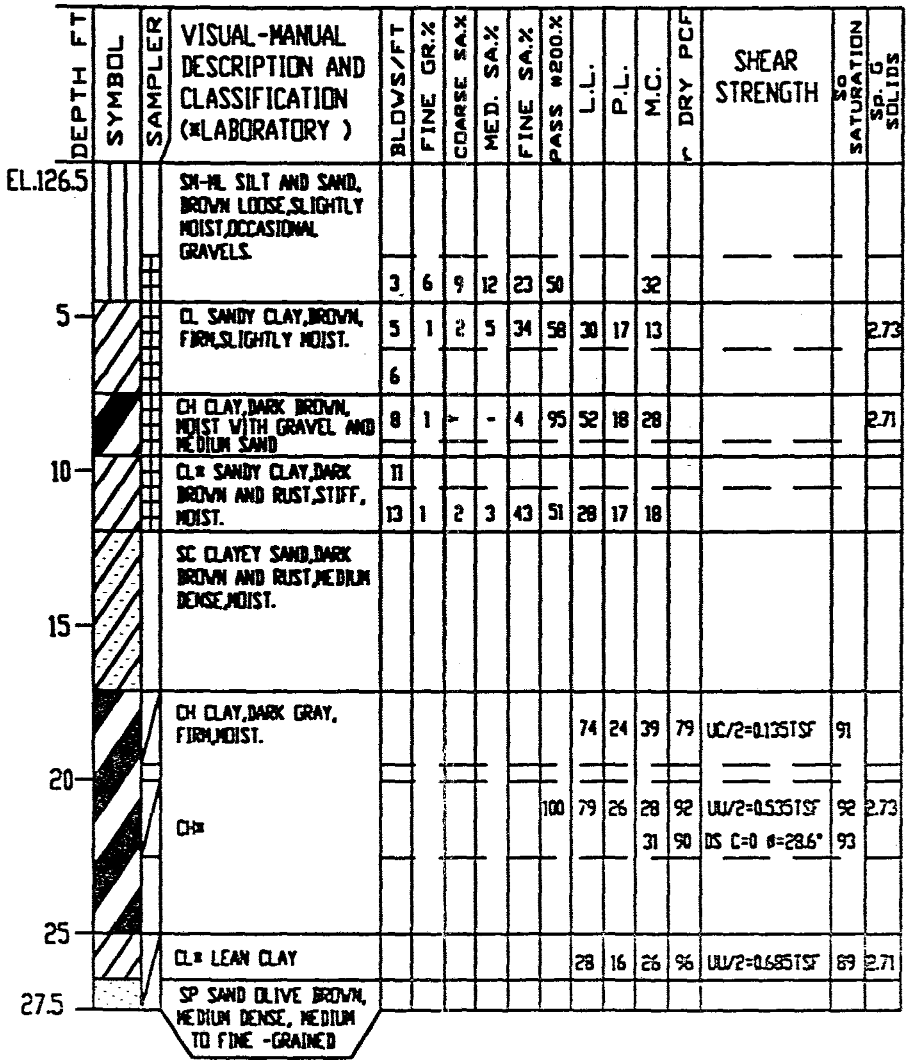
GR-93-2 (28 MAY 1993)



VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200 %	L.L.	P.L.	M.C.	γ DRY PCF	SHEAR STRENGTH	% SATURATION	SP. G SOLIDS
CL-LEAN CLAY, BROWN, SOFT, MOIST													
CL*	2						47	19	28				
CH*MEDIUM STIFF, DARK OLIVE BROWN							53	19	32	83	UC/2=0.39TSF	83	2.75
CH*					2	98	62	21	44 39	77 79	UU/2=0.38TSF UC/2=0.43TSF	92	2.76
CH* OLIVE GRAY													
CH*							53	19	33	88	DS C=0.29TSF ϕ=19.3°	95	2.74

BOTTOM OF BORING

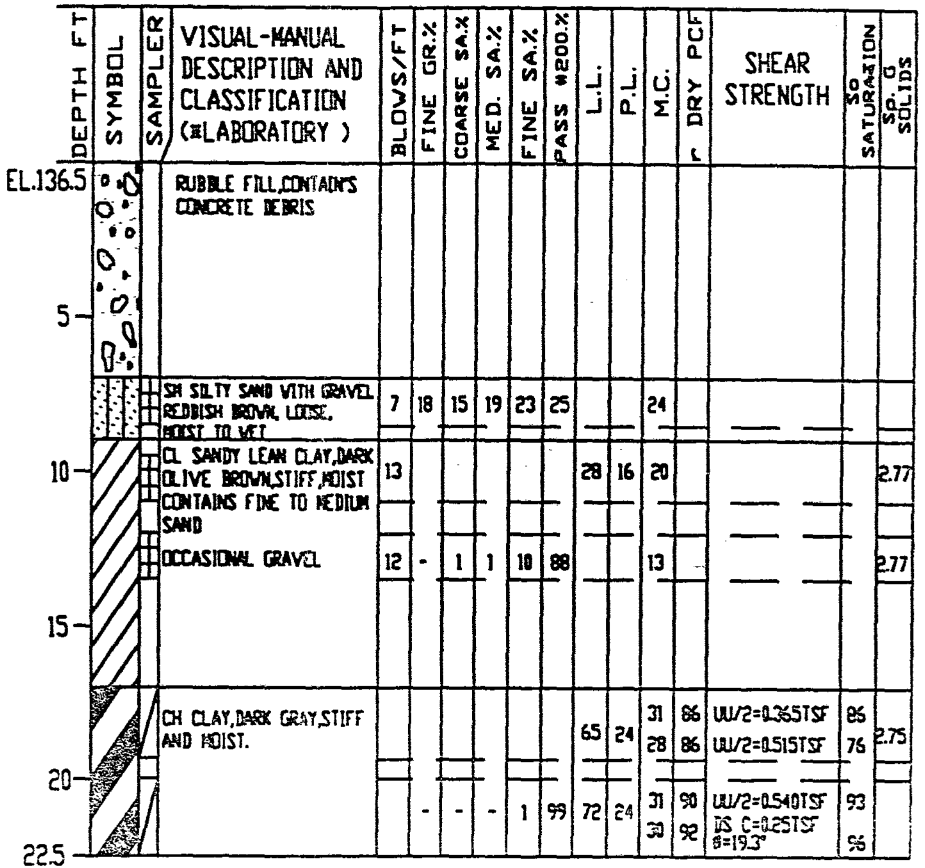
GR-93-3 (27 MAY 1993)



VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200.%	L.L.	P.L.	M.C.	γ DRY PCF	SHEAR STRENGTH	% SATURATION	SP. G SOLIDS
SM-ML SILT AND SAND, BROWN LOOSE, SLIGHTLY MOIST, OCCASIONAL GRAVELS.	3	6	9	12	23	50			32				
CL SANDY CLAY, BROWN, FIRM, SLIGHTLY MOIST.	5	1	2	5	34	58	30	17	13				2.73
	6												
CH CLAY, DARK BROWN, MOIST WITH GRAVEL AND MEDIUM SAND	8	1	-	-	4	95	52	18	28				2.71
CL* SANDY CLAY, DARK BROWN AND RUST, STIFF, MOIST.	11												
	13	1	2	3	43	51	28	17	18				
SC CLAYEY SAND, DARK BROWN AND RUST, MEDIUM DENSE, MOIST.													
CH CLAY, DARK GRAY, FIRM, MOIST.							74	24	39	79	UC/2=0.135TSF	91	
CH*						100	79	26	28 31	92 90	UU/2=0.535TSF DS C=0 ϕ=28.6°	92 93	2.73
CL* LEAN CLAY							28	16	26	96	UU/2=0.685TSF	69	2.71
SP SAND OLIVE BROWN, MEDIUM DENSE, MEDIUM TO FINE -GRAINED													

BOTTOM OF BORING

GR-93-4 (27 MAY 1993)



VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200.%	L.L.	P.L.	M.C.	γ DRY PCF	SHEAR STRENGTH	% SATURATION	SP. G SOLIDS
RUBBLE FILL, CONTAINS CONCRETE DEBRIS													
SM SILTY SAND WITH GRAVEL REDDISH BROWN, LOOSE, MOIST TO WET	7	18	15	19	23	25			24				
CL SANDY LEAN CLAY, DARK OLIVE BROWN, STIFF, MOIST CONTAINS FINE TO MEDIUM SAND	13						28	16	20				2.77
OCCASIONAL GRAVEL	12	-	1	1	10	88			13				2.77
CH CLAY, DARK GRAY, STIFF AND MOIST.							65	24	31 28	86 86	UU/2=0.365TSF UU/2=0.515TSF	85 76	2.75
		-	-	-	1	99	72	24	31 30	90 92	UU/2=0.540TSF DS C=0.25TSF ϕ=19.3°	93 96	

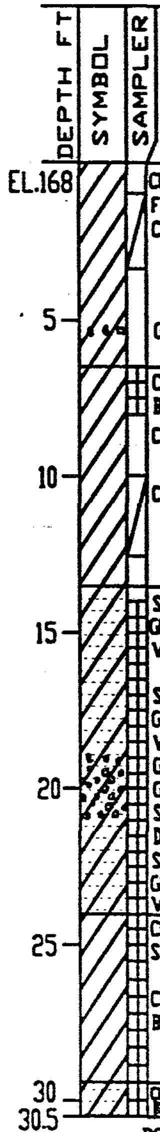
BOTTOM OF BORING

US ARMY ENGINEER DISTRICT SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA
DRAWN BY: SANTA CLARA COUNTY CALIFORNIA
CHECKED BY:

UPPER GUADALUPE RIVER
BORING LOGS

APPROVED: DATE:
SCALE: JOB NO.
DRAWING NO. PLATE: 19
SHEET:

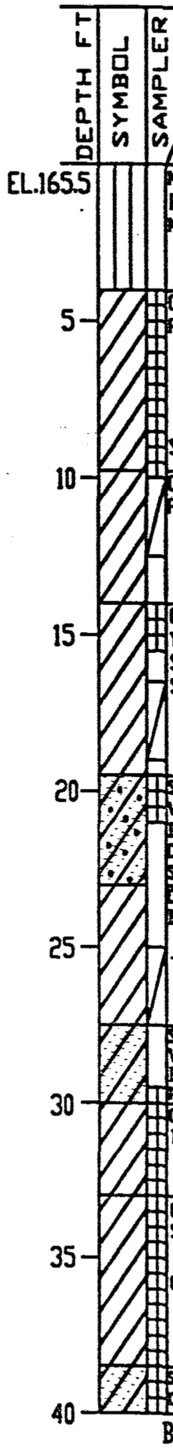
GR-93-7 (26 MAY 1995)



VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200.%	L.L.	P.L.	M.C.	γ DRY PCF	SHEAR STRENGTH	So SATURATION	Sp. G SOLIDS
CL LEAN CLAY,DARK BROWN FIRM													
CL LEAN CLAY							30	17	13 12	87 92	UU/2=0.17TSF UC/2=0.75TSF	35 39	2.75
GRAVELS AT 5.5 FT													
CL SANDY LEAN CLAY, BROWN,STIFF,MOIST.	20	2	4	9	22	63	26	15	22				
CL SANDY CLAY													
CL*							30	12	21 20	99 94	UU/2=0.22TSF UC/2=0.17TSF	78 67	2.75
SC CLAYTY SAND WITH GRAVEL,DARK OLIVE BROWN VERY DENSE	51	28	14	17	18	23	26	13	11				
	49												
SC CLAYTY SAND WITH GRAVEL,DARK OLIVE BROWN VERY DENSE.	79	34	15	22	13	16	27	13	12				
GP-GC POORLY GRADED GRAVEL WITH CLAY AND SAND,DARK OLIVE BROWN, DENSE.	49												
	45												
SC CLAYEY SAND WITH GRAVEL,DARK OLIVE BROWN VERY DENSE.	63	31	15	21	15	18	28	12	14				
	25												
CL LEAN CLAY,BROWN, STIFF,MOIST.	15												
CL LEAN CLAY WITH SAND, BROWN,STIFF	13	1	0	1	18	80	31	13	24				
	13												
CL*SANDY CLAY,DARK OLIVE BROWN,FIRM	7	--	2	1	31	66	29	15	25				2.76

BOTTOM OF BRING

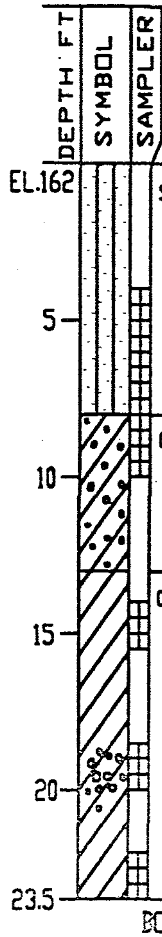
GR-93-6 (25 MAY 1993)



VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200.%	L.L.	P.L.	M.C.	γ DRY PCF	SHEAR STRENGTH	So SATURATION	Sp. G SOLIDS
ML SILT WITH SAND,DARK BROWN,LOOSE,DRY TO MOIST.													
CL*SANDY CLAY,BROWN, FIRM,DRY TO MOIST.	6												
	7	--	--	2	41	57	27	15	15				
	7	--	--	5	21	74	33	18	19				
SC CLAY SAND LENSE,0.2FT	7												
CL LEAN CLAY,DARK BROWN FIRM,DRY TO MOIST.							45	18	19 23	105 99	UU/2=0.710TSF UC/2=0.765TSF	83 84	2.77
BECOMES STIFF.													
CL SANDY LEAN CLAY, YELLOWISH BROWN,VERY STIFF DRY,CONTAINS,FINE SAND.	18	--	--	1	27	72	30	15	11				
							34	17	13				2.78
SP-SC POORLY GRADED SAND WITH CLAY AND GRAVEL,OLIVE BROWN,VERY DENSE,DRY, CONTAINS FINE TO COARSE SAND.	46	43	15	16	12	14	29	14	40				
CL*LEAN CLAY WITH SAND, BROWN,STIFF TO VERY STIFF, DRY TO MOIST.													
▽ GROUND WATER AT 25.34 FT ON 5/27/93		--	--	2	25	73	31	15	22	102	UC/2=0.440TSF		2.50
SP-SC POORLY GRADED SAND WITH CLAY AND GRAVEL, BROWN,MEDIUM DENSE,DRY TO MOIST.													
CL SANDY CLAY,BROWN,STIFF TO VERY SIFF,MOIST.	12	9	3	3	17	68	29	16	24				
	20	1	3	16	16	64	25	14	25				
CL*LEAN CLAY,OLIVE BROWN, STIFF TO VERY STIFF,MOIST.	8												
	9	--	--	--	6	94	38	13	29				
CL SANDY LEAN CLAY,STIFF.	15												
	15						25	12	21				
SP-SC POORLY GRADED SAND WITH CLAY AND GRAVELS,BROWN,DENSE	44												

BOTTOM OF BORING

GR-93-5 (28 MAY 1993)



VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200.%	L.L.	P.L.	M.C.	γ DRY PCF	SHEAR STRENGTH	Sc SATURATION	Sp. G SOLIDS
SM SILTY SAND,DARK BROWN,VERY LOOSE, DRY TO SLIGHTLY MOIST.	2												
	5	1	1	4	56	38			20				2.73
	13	20	7	11	21	41	25	16	12				2.76
GP-GC POORLY GRADED GRAVEL WITH CLAY, DARK OLIVE BROWN MEDIUM DENSE,SLIGHTY MOIST.	22												
CL SANDY LEAN CLAY, BROWN,STIFF,MOIST, CONTAIN'S FINE SAND	11						36	14	38				2.77
WITH GRAVEL	10	19	6	7	13	55	28	14	29				
	13												

BOTTOM OF BORING

US ARMY ENGINEER DISTRICT SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

DRAWN BY: SANTA CLARA COUNTY CALIFORNIA

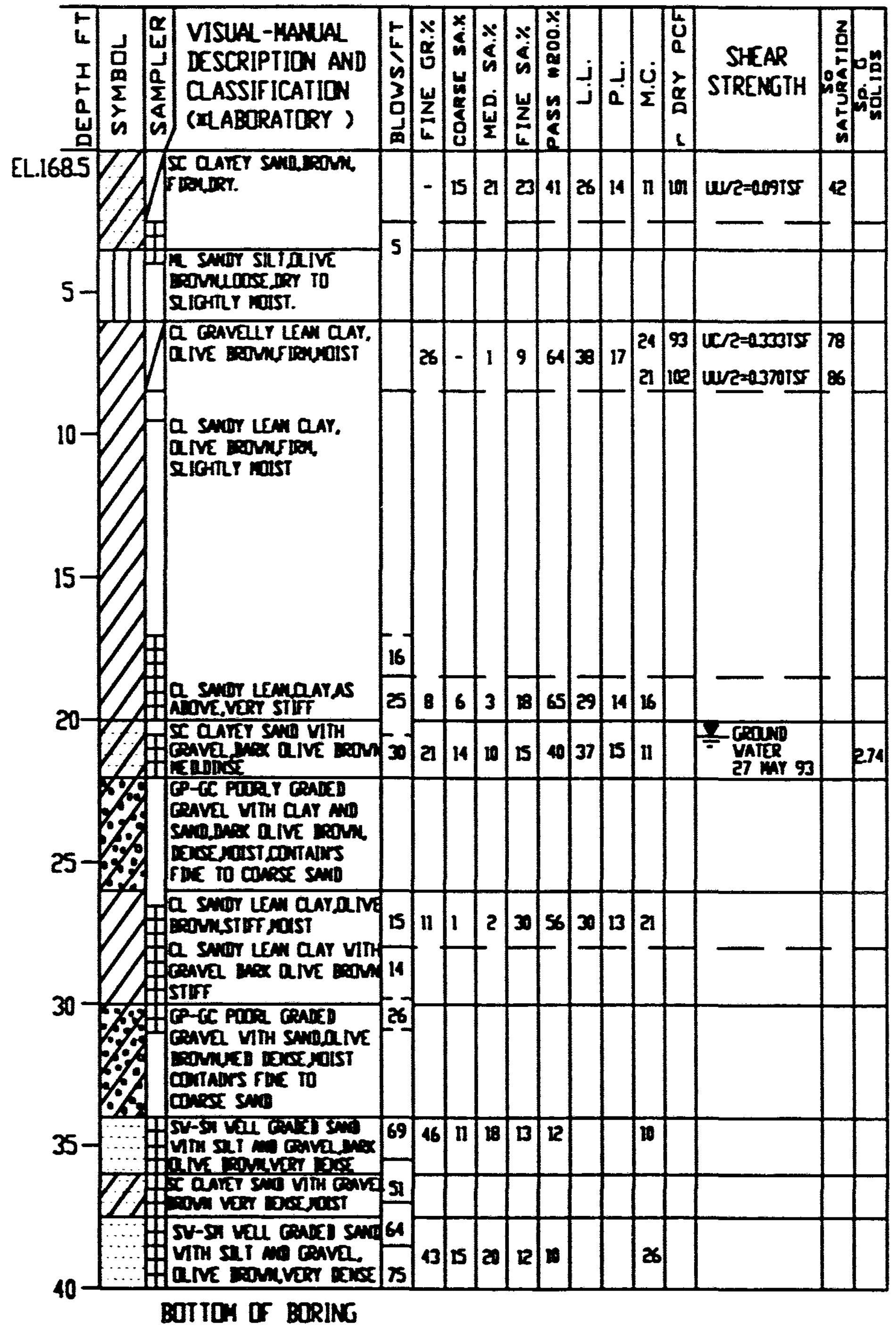
CHECKED BY:

UPPER GUADALUPE RIVER
BORING LOGS

APPROVED: DATE:

SCALE: JOB NO.
DRAWING NO. PLATE: 15
SHEET:

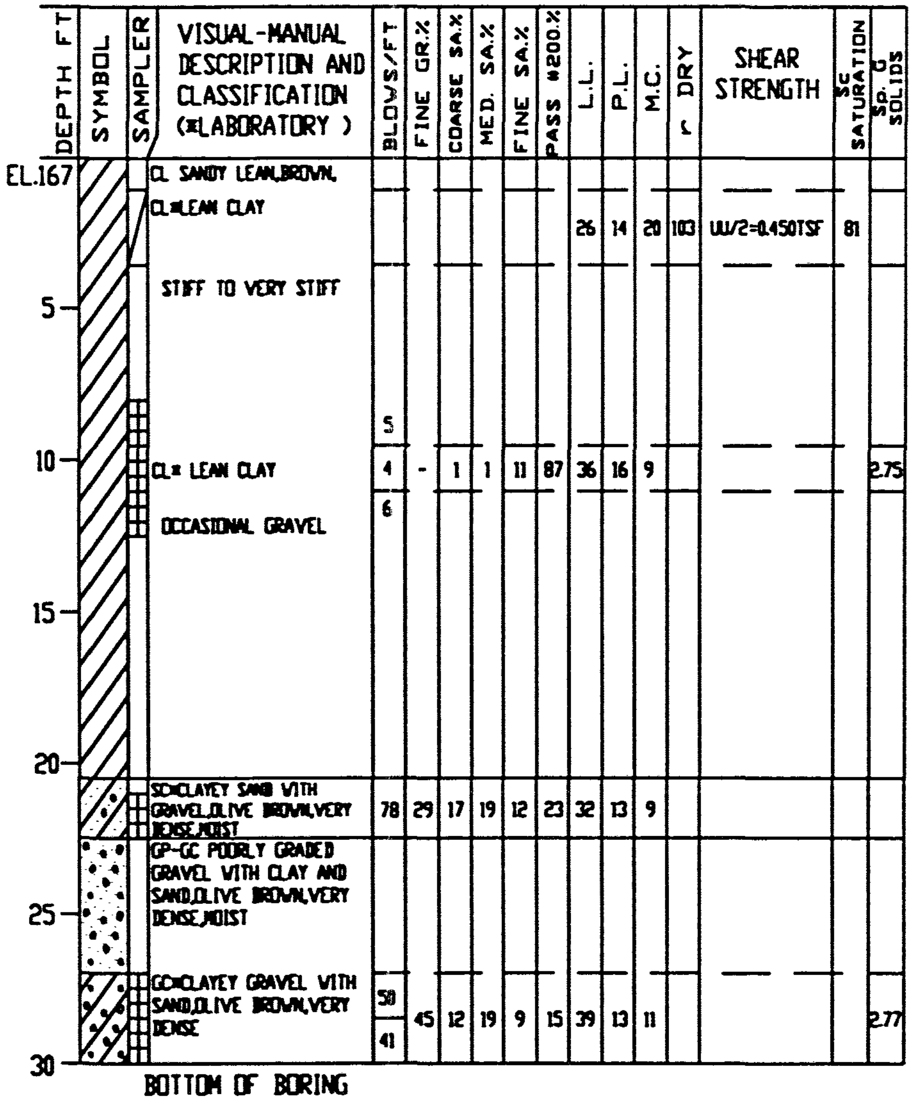
RC-93-1 (25 MAY 1993)



DEPTH FT	VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200.%	L.L.	P.L.	M.C.	γ DRY PCF	SHEAR STRENGTH	S% SATURATION	Sp. G SOLIDS
EL.168.5	SC CLAYEY SAND,BROWN, FIRM,DRY.		-	15	21	23	41	26	14	11	101	UU/2=0.09TSF	42	
	ML SANDY SILT,OLIVE BROWN,LOOSE,DRY TO SLIGHTLY MOIST.	5												
	CL GRAVELLY LEAN CLAY, OLIVE BROWN,FIRM,MOIST		26	-	1	9	64	38	17	24 21	93 102	UC/2=0.333TSF UU/2=0.370TSF	78 86	
	CL SANDY LEAN CLAY, OLIVE BROWN,FIRM, SLIGHTLY MOIST	16												
	CL SANDY LEAN,CLAY,AS ABOVE,VERY STIFF	25	8	6	3	18	65	29	14	16				
	SC CLAYEY SAND WITH GRAVEL,DARK OLIVE BROWN MED.DENSE	30	21	14	10	15	40	37	15	11		GROUND WATER 27 MAY 93		2.74
	GP-GC POORLY GRADED GRAVEL WITH CLAY AND SAND,DARK OLIVE BROWN, DENSE,MOIST,CONTAIN'S FINE TO COARSE SAND													
	CL SANDY LEAN CLAY,OLIVE BROWN,STIFF,MOIST	15	11	1	2	30	56	30	13	21				
	CL SANDY LEAN CLAY WITH GRAVEL DARK OLIVE BROWN STIFF	14												
	GP-GC POORL GRADED GRAVEL WITH SAND,OLIVE BROWN,MED DENSE,MOIST CONTAIN'S FINE TO COARSE SAND	26												
	SW-SM WELL GRADED SAND WITH SILT AND GRAVEL,DARK OLIVE BROWN,VERY DENSE	69	46	11	18	13	12			10				
	SC CLAYEY SAND WITH GRAVEL BROWN VERY DENSE,MOIST	51												
40	SW-SM WELL GRADED SAND WITH SILT AND GRAVEL, OLIVE BROWN,VERY DENSE	64 75	43	15	20	12	18			26				

BOTTOM OF BORING

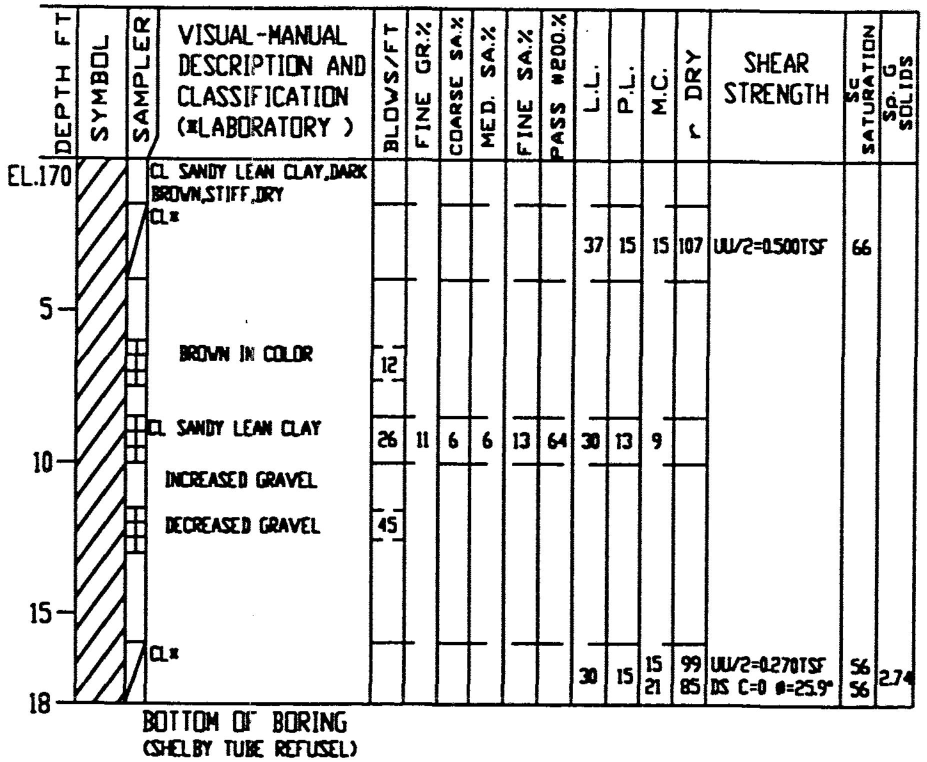
RC-93-2 (26 MAY 1993)



DEPTH FT	VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200.%	L.L.	P.L.	M.C.	γ DRY	SHEAR STRENGTH	S% SATURATION	Sp. G SOLIDS
EL.167	CL SANDY LEAN,BROWN,													
	CL*LEAN CLAY							26	14	20	103	UU/2=0.450TSF	81	
	STIFF TO VERY STIFF	5												
	CL* LEAN CLAY	4	-	1	1	11	87	36	16	9				2.75
	OCCASIONAL GRAVEL	6												
	SC*CLAYEY SAND WITH GRAVEL,OLIVE BROWN,VERY DENSE,MOIST	78	29	17	19	12	23	32	13	9				
	GP-GC POORLY GRADED GRAVEL WITH CLAY AND SAND,OLIVE BROWN,VERY DENSE,MOIST													
30	GC*CLAYEY GRAVEL WITH SAND,OLIVE BROWN,VERY DENSE	50 41	45	12	19	9	15	39	13	11				2.77

BOTTOM OF BORING

RC-93-3 (26 MAY 1993)



DEPTH FT	VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS #200.%	L.L.	P.L.	M.C.	γ DRY	SHEAR STRENGTH	S% SATURATION	Sp. G SOLIDS
EL.170	CL SANDY LEAN CLAY,DARK BROWN,STIFF,DRY													
	CL*							37	15	15	107	UU/2=0.500TSF	66	
	BROWN IN COLOR	12												
	CL SANDY LEAN CLAY	26	11	6	6	13	64	30	13	9				
	INCREASED GRAVEL													
	DECREASED GRAVEL	45												
18	CL*							30	15	15 21	99 85	UU/2=0.270TSF DS C=0 ø=25.9°	56 56	2.74

BOTTOM OF BORING
(SHELBY TUBE REFUSEL)

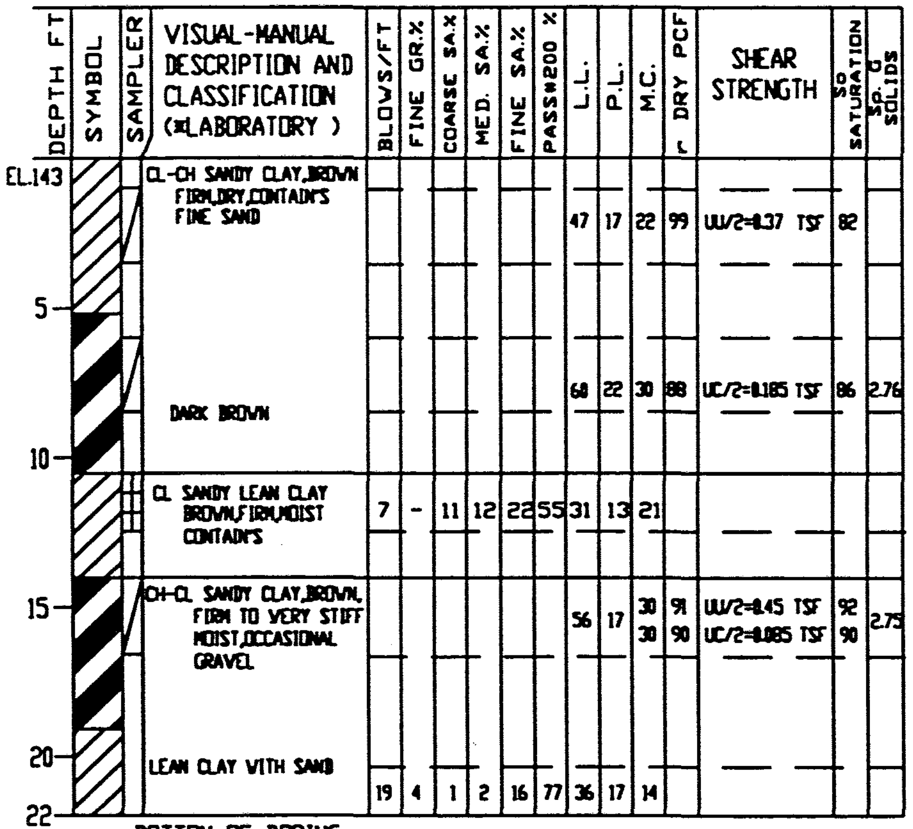
US ARMY ENGINEER DISTRICT SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

DRAWN BY:
SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
BORING LOGS

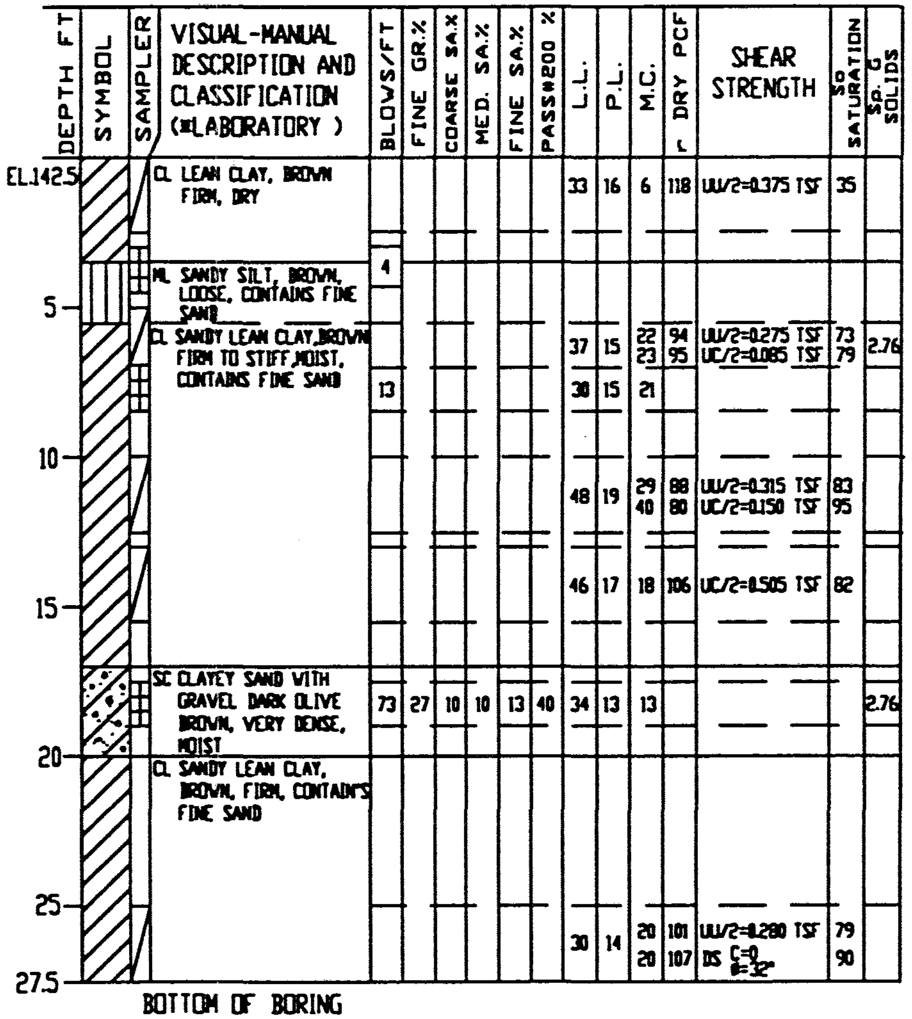
CHECKED BY:
APPROVED:
DATE:
SCALE:
JOB NO:
DRAWING NO:
PLATE: 16
SHEET:

CC-93-1 (27 MAY 1993)



DEPTH FT	VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS#200 %	L.L.	P.L.	M.C.	r DRY PCF	SHEAR STRENGTH	S% SATURATION	Sp. G SOLIDS
EL.143	CL-CH SANDY CLAY,BROWN FIRM,DRY,CONTAIN'S FINE SAND							47	17	22	99	UU/2=0.37 TSF	82	
5	DARK BROWN							68	22	30	88	UC/2=0.185 TSF	86	2.76
10	CL SANDY LEAN CLAY BROWN,FIRM,MOIST CONTAIN'S	7	-	11	12	22	55	31	13	21				
15	CH-CL SANDY CLAY,BROWN, FIRM TO VERY STIFF MOIST,OCCASIONAL GRAVEL							56	17	30 30	91 90	UU/2=0.45 TSF UC/2=0.085 TSF	92 90	2.75
20	LEAN CLAY WITH SAND	19	4	1	2	16	77	36	17	14				
22	BOTTOM OF BORING													

CC-93-2 (27 MAY 1993)



DEPTH FT	VISUAL-MANUAL DESCRIPTION AND CLASSIFICATION (*LABORATORY)	BLOWS/FT	FINE GR.%	COARSE SA.%	MED. SA.%	FINE SA.%	PASS#200 %	L.L.	P.L.	M.C.	r DRY PCF	SHEAR STRENGTH	S% SATURATION	Sp. G SOLIDS
EL.142.5	CL LEAN CLAY, BROWN FIRM, DRY							33	16	6	118	UU/2=0.375 TSF	35	
	ML SANDY SILT, BROWN, LOOSE, CONTAINS FINE SAND	4												
5	CL SANDY LEAN CLAY,BROWN FIRM TO STIFF,MOIST, CONTAINS FINE SAND							37	15	22 23	94 95	UU/2=0.275 TSF UC/2=0.085 TSF	73 79	2.76
		13						38	15	21				
10								48	19	29 40	88 80	UU/2=0.315 TSF UC/2=0.150 TSF	83 95	
15								46	17	18	106	UC/2=0.505 TSF	82	
	SC CLAYEY SAND WITH GRAVEL DARK OLIVE BROWN, VERY DENSE, MOIST	73	27	10	10	13	40	34	13	13				2.76
20	CL SANDY LEAN CLAY, BROWN, FIRM, CONTAIN'S FINE SAND													
25								30	14	20 20	101 107	UU/2=0.280 TSF DS C=0 Ø=32°	79 90	
27.5	BOTTOM OF BORING													

LEGEND:

SM	SILTY SAND
CL	CLAY (LOW PLASTICITY)
CH	CLAY (HIGH PLASTICITY)
SC	CLAYEY SAND
SP	UNIFORM SAND
ML	SILT (LOW PLASTICITY)
NP	NON PLASTICITY
Gr	GRAVEL, UNIFIED SOIL CLASSIFICATION SYSTEM
Sa	SAND, UNFIED SOIL CLASSIFICATION SYSTEM
Fi	FINES (PERECNT PASS NO.200 SIEVE)
N	NUMBER OF BLOW COUNTS PER FOOT USING THE STANDARD PENETROTION TEST PROCEDURE IN ACCORDANCE WITH ASTM STANDARD D1586 UNLESS OTHERWISE NOTED
LL	LOQUID LIMIT
PL	PLASTIC LIMIT
MC	MOISTURE CONTENT
DRY	DRY UNIT MASS (POUND PER CUBIC FOOT)
UU	UNCONSOLIDATION, UNDRAINED, TRIAXIAL SHEAR STRENGTH (TON PER SQUARE FOOT)
CU	CONSOLIDATION, UNDRAINED, TRIAXIAL SHEAR STRENGTH (TON PER SQUARE FOOT)
LVS	LABORATORY VANE SHEAR STRENGTH
UC	UNCONFINED COMPRESSIVE STRENGTH

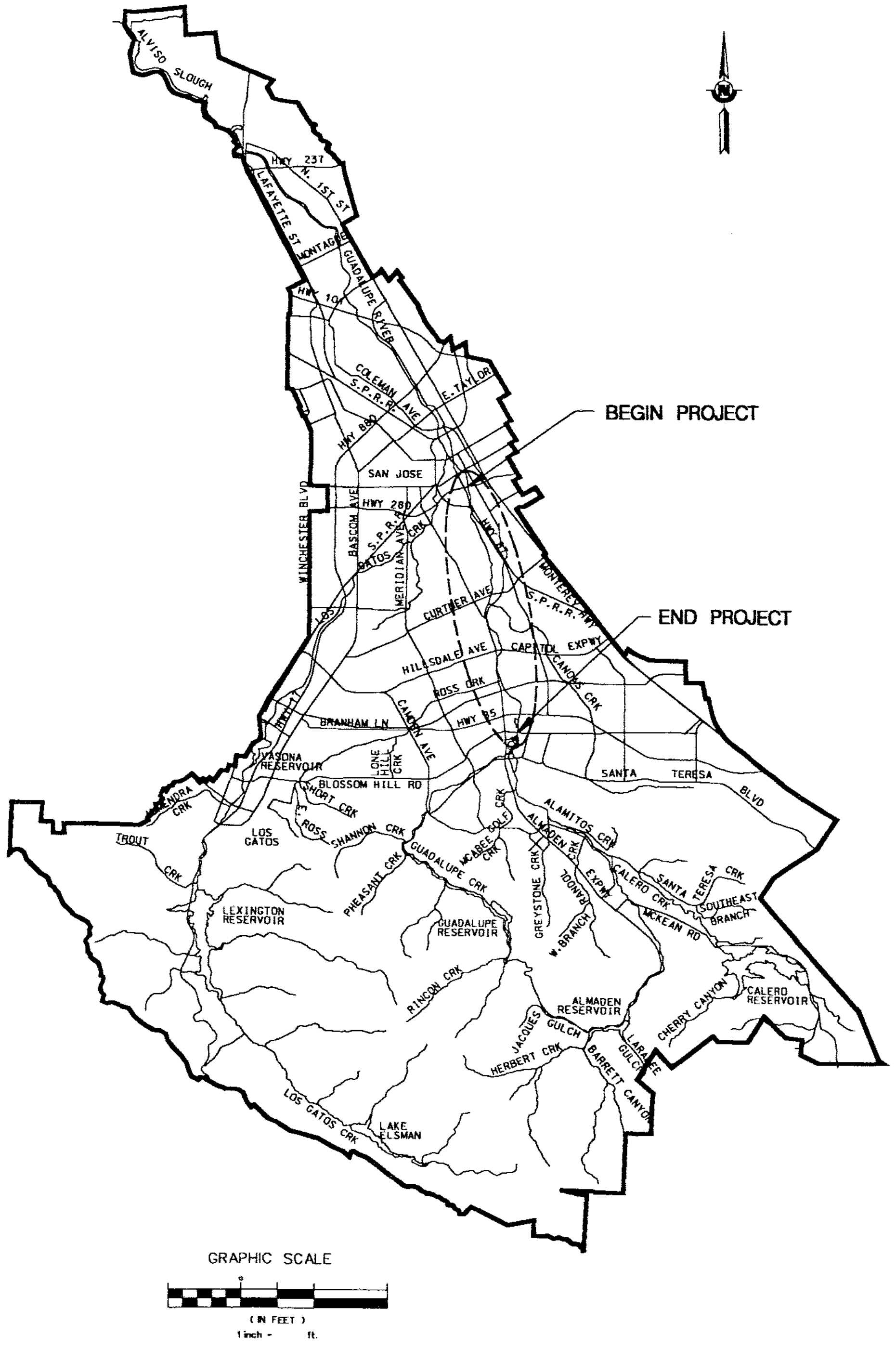
US ARMY ENGINEER DISTRICT SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

DRAWN BY: SANTA CLARA COUNTY CALIFORNIA

UPPER GUADALUPE RIVER
BORING LOGS

CHECKED BY:
APPROVED:
DATE:
SCALE:
JOB NO:
DRAWING NO:
PLATE: 14
SHEET:

FEASIBILITY DESIGN
UPPER GUADALUPE RIVER FLOOD CONTROL PROJECT
FROM INTERSTATE 280 TO BLOSSOM HILL ROAD
SANTA CLARA COUNTY, CALIFORNIA



LOCATION MAP

LIST OF DRAWINGS

SHEET NO.	DESCRIPTION
T-1	TITLE SHEET, LOCATION MAP, LIST OF DRAWINGS, ABBREVIATIONS & LEGENDS
C-1	PLAN AND PROFILE STA. 739+60 TO STA. 761+80
C-2	PLAN AND PROFILE STA. 761+80 TO STA. 790+00
C-3	PLAN AND PROFILE STA. 790+00 TO STA. 816+30
C-4	BYPASS CHANNEL PROFILE STA. 790+10 TO STA. 795+00
C-5	PLAN AND PROFILE STA. 816+30 TO STA. 844+85
C-6	PLAN AND PROFILE STA. 844+85 TO STA. 866+05
C-7	CANOAS CREEK STA. 0+00 TO STA. 28+85
C-8	PLAN AND PROFILE STA. 866+05 TO STA. 887+80
C-9	PLAN AND PROFILE STA. 887+80 TO STA. 911+75
C-10	PLAN AND PROFILE STA. 911+75 TO STA. 937+60
C-11	PLAN AND PROFILE STA. 937+60 TO STA. 963+35
C-12	ROSS CREEK STA. 0+00 TO STA. 28+55
C-13	ROSS CREEK STA. 28+55 TO STA. 52+50
C-14	PLAN AND PROFILE STA. 963+35 TO STA. 992+00
C-15	PLAN AND PROFILE STA. 992+00 TO STA. 1019+30
C-16	PLAN AND PROFILE STA 1019+30 TO STA. 1034+00
C-17	TYPICAL SECTIONS STA. 751+85 TO STA. 780+70
C-18	TYPICAL SECTIONS STA. 782+25± TO STA. 806+00±
C-19	TYPICAL SECTIONS STA. 810+00± TO STA. 821+50±
C-20	TYPICAL SECTIONS STA. 822+00± TO STA. 844+50±
C-21	TYPICAL SECTIONS STA. 858+20± TO STA. 885+50±
C-22	TYPICAL SECTIONS STA. 888+00± TO STA. 917+50±
C-23	TYPICAL SECTIONS STA. 917+00± TO STA. 948+00±
C-24	TYPICAL SECTIONS STA. 949+60 TO STA. 968+20
C-25	TYPICAL SECTIONS STA. 952+80± TO STA. 1012+00±
C-26	CANOAS CREEK & ROSS CREEK TYPICAL SECTIONS
C-27	PLAN DETAIL BEGINNING OF PROJECT & BYPASS CHANNEL
C-28	PLAN DETAIL ELKS LODGE OPEN BYPASS
C-29	PLAN DETAIL PINE AVENUE ISLAND
C-30	PLAN DETAIL MALONE ROAD ISLAND
C-31	PLAN DETAIL ROSS CREEK CONFLUENCE
C-32	SECTIONS AND DETAIL ROSS CREEK CONFLUENCE
C-33	VORTEX ROCK WEIR DETAIL PLAN, PROFILE & SECTION
C-34	PLANTING BOX PLAN, SECTIONS & DETAIL
C-35	SITES NEEDING EROSION REPAIR
C-36	BRIDGES AND CULVERTS STA. 738+80 TO STA. 856+50
C-37	BRIDGES AND CULVERTS STA. 870+00 TO STA. 987+70
C-38	BRIDGES AND CULVERTS CANOAS CREEK AND ROSS CREEK

ABBREVIATIONS

%	PERCENT
1%	100 YEAR FLOOD
10%	10 YEAR FLOOD
ACM	ARTICULATED CONCRETE MAT
℄	CENTERLINE
CFS	CUBIC FEET PER SECOND
CIP	CAST IRON PIPE
CMP	CORRUGATED METAL PIPE
CONC	CONCRETE
CY	CUBIC YARDS
D/S	DOWNSTREAM
DEP	DEPRESSED
DIP	DUCTILE IRON PIPE
E	EAST
EL	ELEVATION
EP	EDGE OF PAVEMENT
EASM'T	EASEMENT
EXIST	EXISTING
EXPWY	EXPRESSWAY
FH	FIRE HYDRANT
FOC	FACE OF CURB
FT (')	FEET (FOOT)
FWY	FREEWAY
HWY	HIGHWAY
JP	JOINT POLE
MAINT	MAINTENANCE
MAX	MAXIMUM
MH	MANHOLE
MIN	MINIMUM
N	NORTH
NTS	NOT TO SCALE
OH	OVERHEAD
PG&E	PACIFIC GAS AND ELECTRIC
Q	DESIGN DISCHARGE (CUBIC FEET PER SECOND)
RCB	REINFORCED CONCRETE BOX
RCP	REINFORCED CONCRETE PIPE
RD	ROAD
REVEG	REVEGETATION
R/W	RIGHT OF WAY
S	SOUTH OR SLOPE
SCVWD	SANTA CLARA VALLEY WATER DISTRICT
SD	STORM DRAIN
SJWW	SAN JOSE WATER WORKS
SPRR	SOUTHERN PACIFIC RAILROAD
SRA	SHADED RIVERINE AQUATIC
SS	SANITARY SEWER
STA	STATION
TEMP	TEMPORARY
TOB	TOP OF BANK
TYP	TYPICAL
UPRR	UNION PACIFIC RAILROAD
U/S	UPSTREAM
VEG	VEGETATION
W	WEST
X-SECTION	CROSS SECTION
YR	YEAR

LEGEND

——	CREEK CENTERLINE
——	PROPOSED SCVWD R/W LINE
——	EXISTING SCVWD R/W LINE
——	EXISTING R/W LINE (NON SCVWD PROPERTY)
— SS —	SANITARY SEWER LINE
— SD —	STORM DRAIN LINE
— W —	WATER LINE
— OH —	OVERHEAD LINE
——	PROPOSED LINE (TYP)
——	EXISTING LINE (TYP)
▽	WATER SURFACE PROFILE (1% FLOW)
▼	WATER SURFACE PROFILE (10% FLOW AND OTHER FLOWS)
▦	PROPOSED STEPPED GABION LINING
▤	PROPOSED MATTRESS GABION LINING
▨	PROPOSED ROCK LINING
▒	PROPOSED CONCRETE LINING
▽ ▽	EXISTING SLOPE OF BANK
▼ ▼	PROPOSED SLOPE OF BANK
═	PROPOSED GATED FENCE FOR MAINTENANCE ROAD
▨	EXISTING ROCK LINING
▒	EXISTING CONCRETE LINING

REFERENCE INFORMATION AND NOTES

1. All cross sections or typical sections shown are looking upstream.
2. All proposed and existing maintenance roads shall be paved with minimum 6" crushed rock.
3. All existing concrete rubble in channel inverts shall be removed wherever possible.
4. All proposed levees and floodwalls shall be constructed at a minimum 3.5 feet above the 1% WSEL.
5. All elevations are based on NGVD 1929.

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

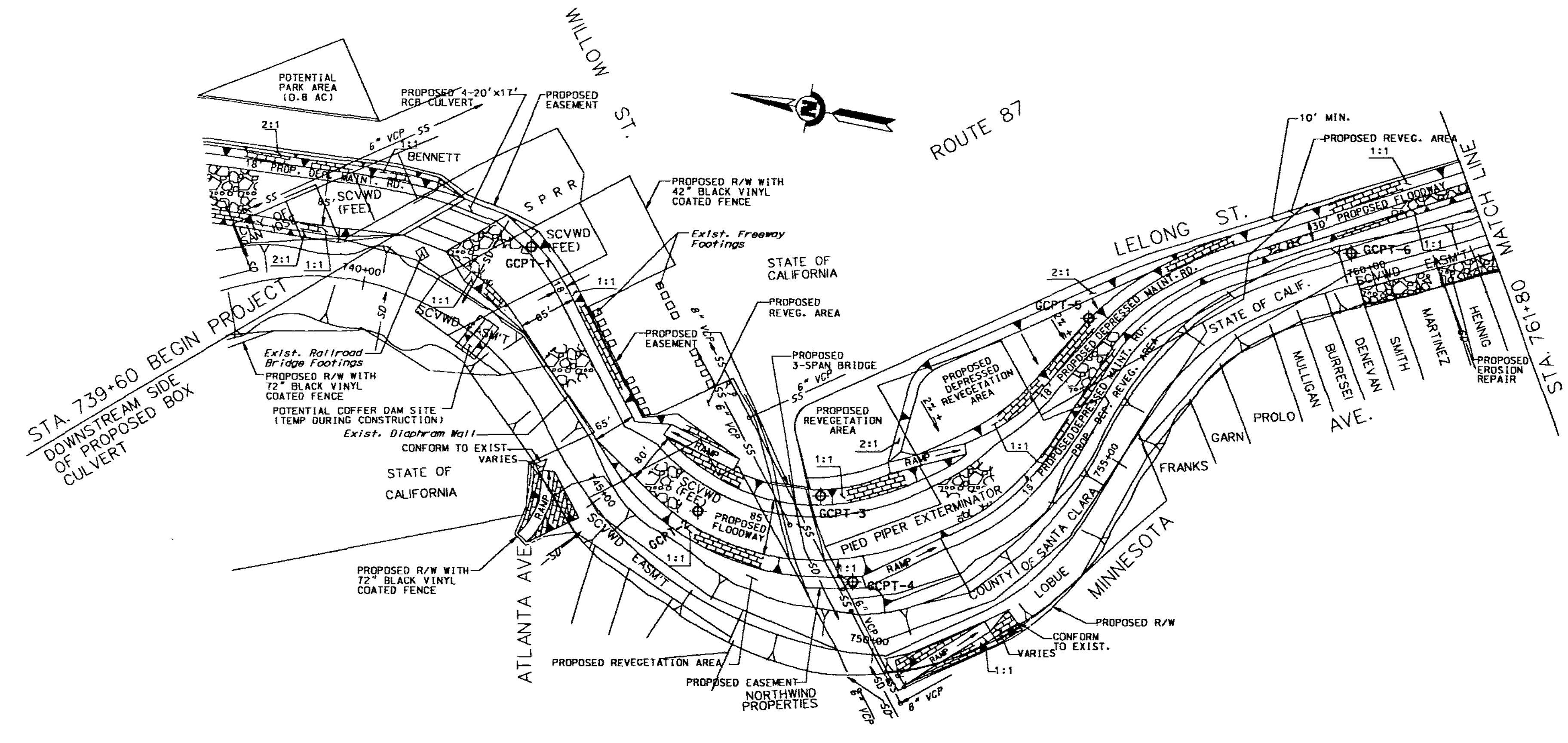
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
TITLE SHEET, LOCATION MAP
LIST OF DRAWINGS
ABBREVIATIONS & LEGENDS

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

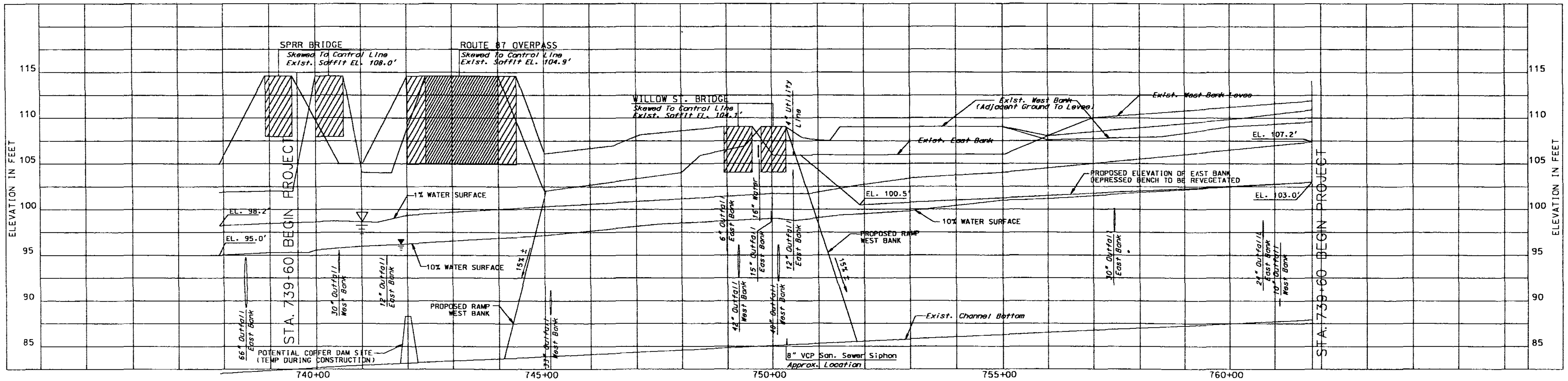
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: 1" = 40'
JOB NO.:
DRAWING NUMBER
SHEET: T-1



PLAN

SCALE: 1"=100'



PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

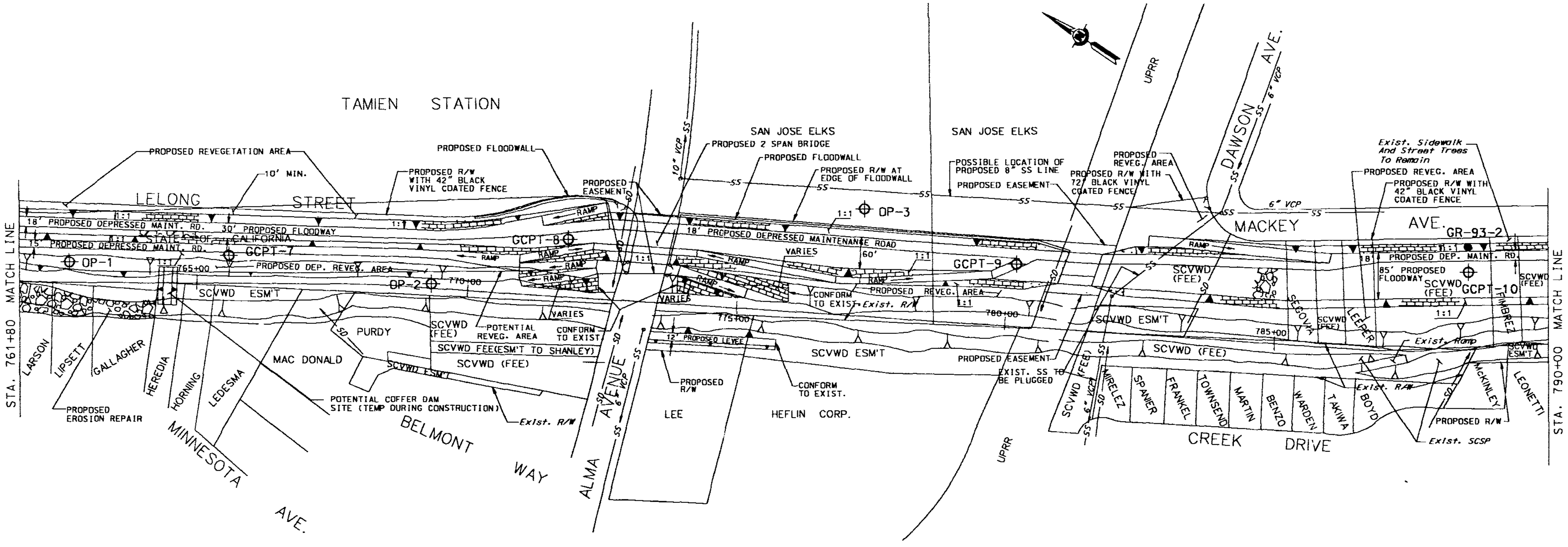
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA. 739+60 TO STA. 761+80

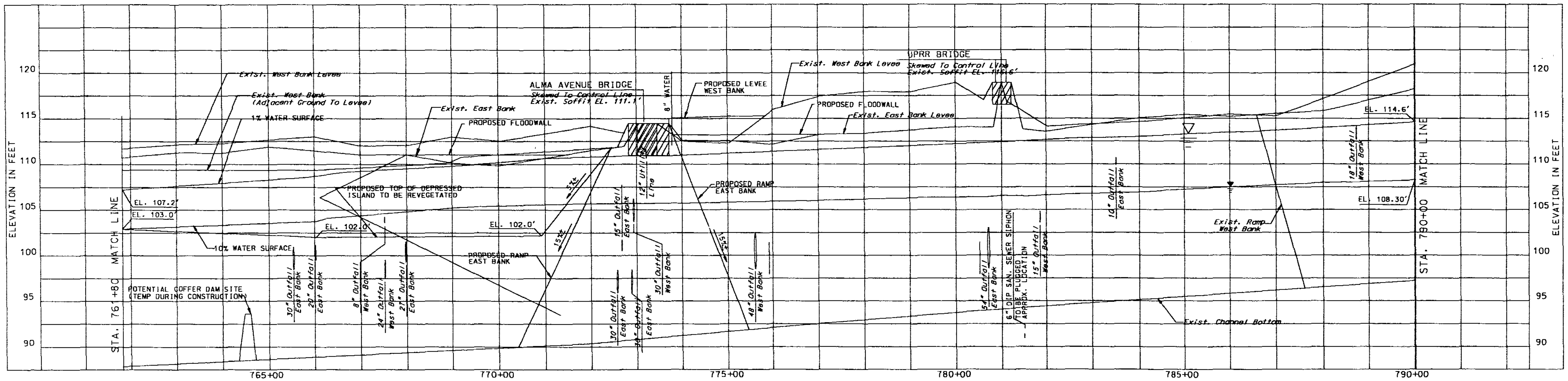
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-1



PLAN
SCALE: 1"=100'



PROFILE
SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

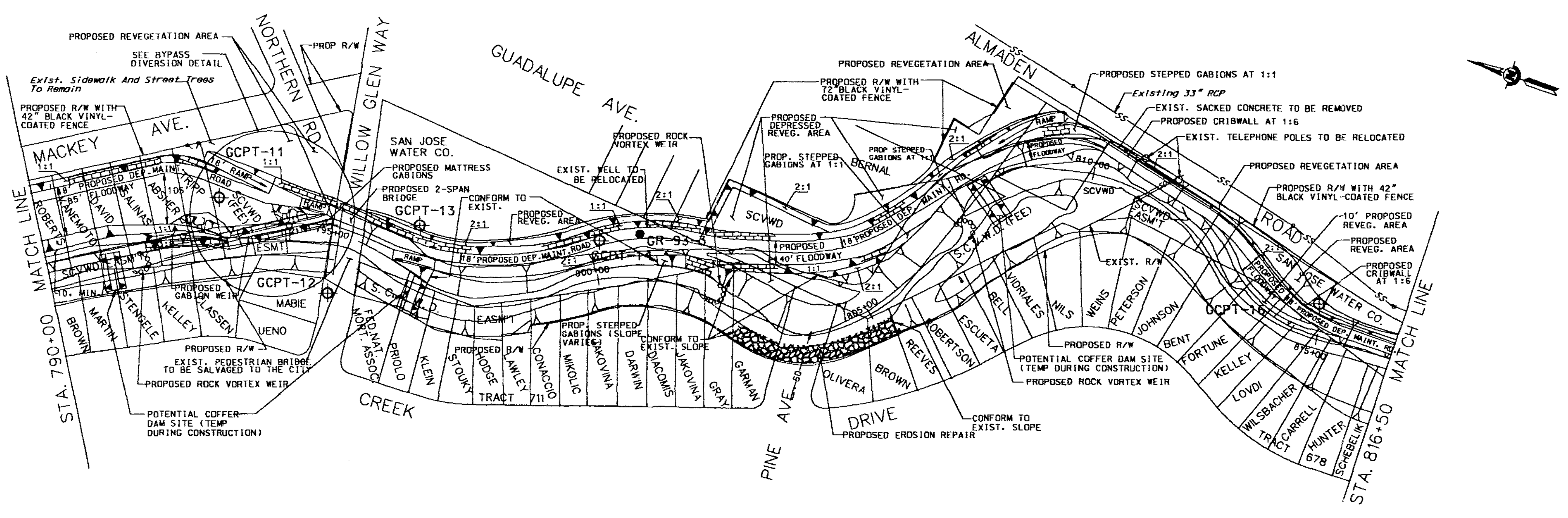
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 761+80 TO STA 790+00

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

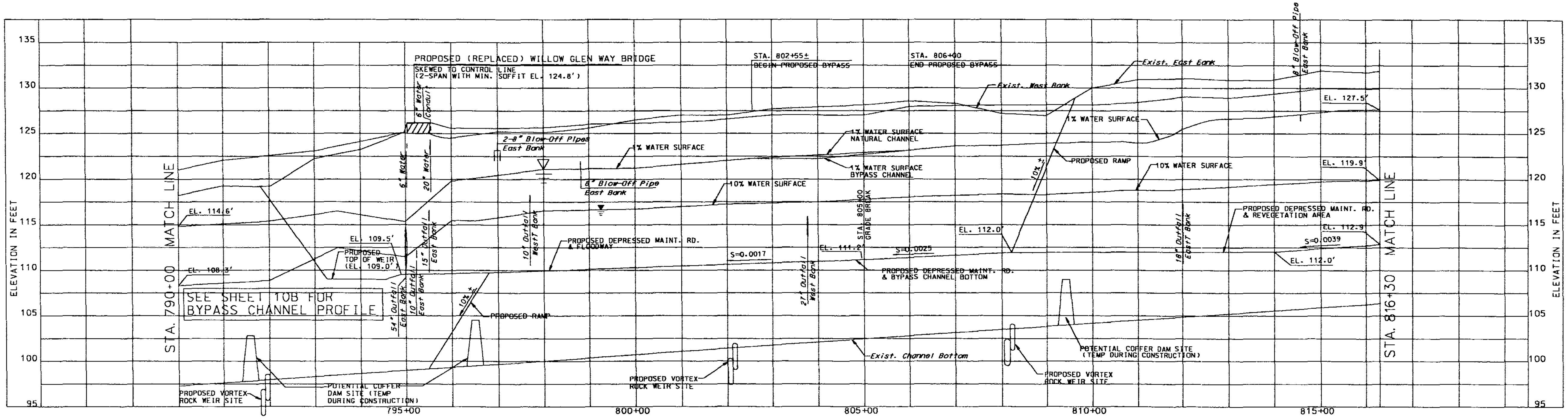
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-2


PROPOSED REVEGETATION AREA
SEE BYPASS DIVERSION DETAIL
Exist. Sidewalk And Street Trees To Remain
PROPOSED R/W WITH 42" BLACK VINYL-COATED FENCE
MACKEY AVE.
NORTHERN RD.
WILLOW GLEN WAY
PROP R/W
GUADALUPE AVE.
SAN JOSE WATER CO.
PROPOSED MATTRESS GABIONS
PROPOSED 2-SPAN BRIDGE
CONFORM TO EXIST.
GCPT-11
GCPT-12
GCPT-13
GCPT-16
GR-93
EXIST. WELL TO BE RELOCATED
PROPOSED ROCK VORTEX WEIR
PROPOSED REVEG. AREA
PROPOSED DEPRESSED REVEG. AREA
PROP. STEPPED GABIONS AT 1:1
PROP STEPPED GABIONS AT 1:1
BERNAL
SCVWD
40' FLOODWAY
18' PROPOSED DEP. MAINT. ROAD
PROPOSED R/W WITH 72" BLACK VINYL-COATED FENCE
ALMADEN
PROPOSED STEPPED GABIONS AT 1:1
Existing 33" RCP
EXIST. SACKED CONCRETE TO BE REMOVED
PROPOSED CRIBWALL AT 1:6
EXIST. TELEPHONE POLES TO BE RELOCATED
PROPOSED R/W WITH 42" BLACK VINYL-COATED FENCE
10' PROPOSED REVEG. AREA
PROPOSED REVEG. AREA
PROPOSED CRIBWALL AT 1:6
ROAD
SAN JOSE WATER CO.
EXIST. R/W
PROPOSED R/W
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
PROPOSED ROCK VORTEX WEIR
CONFORM TO EXIST. SLOPE
PROPOSED EROSION REPAIR
PROP. STEPPED GABIONS (SLOPE VARIES)
PROPOSED GABION WEIR
EXIST. PEDESTRIAN BRIDGE TO BE SALVAGED TO THE CITY
PROPOSED ROCK VORTEX WEIR
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
MATCH LINE STA. 790+00
MATCH LINE STA. 816+50
CREEK
PINE AVE.
DRIVE
TRACT 711
TRACT 678
795+00
800+00
805+00
810+00
815+00

PLAN

SCALE: 1"=100'


135
130
125
120
115
110
105
100
95
ELEVATION IN FEET
STA. 790+00 MATCH LINE
STA. 816+30 MATCH LINE
PROPOSED (REPLACED) WILLOW GLEN WAY BRIDGE
SKEWED TO CONTROL LINE (2-SPAN WITH MIN. SOFFIT EL. 124.8')
STA. 802+55± BEGIN PROPOSED BYPASS
STA. 806+00 END PROPOSED BYPASS
Exist. East Bank
Exist. West Bank
8" Blow-Off Pipe East Bank
EL. 127.5'
1% WATER SURFACE
2-8" Blow-Off Pipes East Bank
1% WATER SURFACE
1% WATER SURFACE NATURAL CHANNEL
1% WATER SURFACE BYPASS CHANNEL
PROPOSED RAMP
10% WATER SURFACE
EL. 119.9'
6" Blow-Off Pipe East Bank
10% WATER SURFACE
STA. 805+00 GRADE BREAK
EL. 114.6'
PROPOSED DEPRESSED MAINT. RD. & REVEGETATION AREA
EL. 112.9'
S=0.0039
EL. 112.0'
EL. 112.0'
EL. 111.2'
S=0.0025
S=0.0017
EL. 109.5'
PROPOSED TOP OF WEIR (EL. 109.0')
PROPOSED DEPRESSED MAINT. RD. & FLOODWAY
PROPOSED DEPRESSED MAINT. RD. & BYPASS CHANNEL BOTTOM
EL. 108.3'
SEE SHEET 10B FOR BYPASS CHANNEL PROFILE
6" Water
6" Water Conduit
20" Water
54" Outfall East Bank
10" Outfall East Bank
15" Outfall East Bank
10" Outfall West Bank
27" Outfall West Bank
18" Outfall East Bank
10%
PROPOSED RAMP
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
Exist. Channel Bottom
PROPOSED VORTEX ROCK WEIR SITE
PROPOSED VORTEX ROCK WEIR SITE
PROPOSED VORTEX ROCK WEIR SITE
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
795+00
800+00
805+00
810+00
815+00

PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

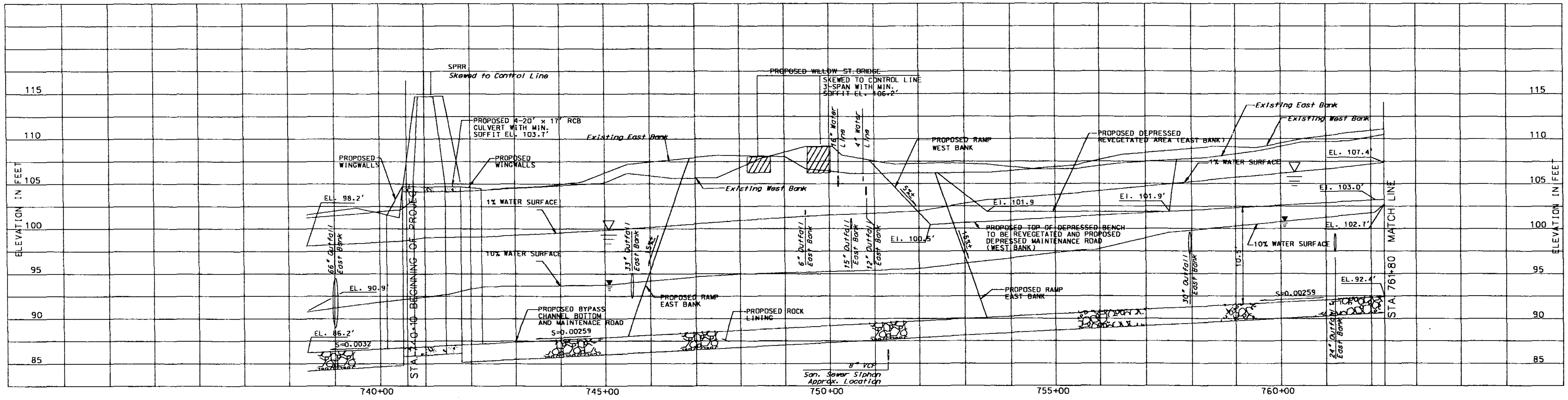
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 790+00 TO STA 816+30

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

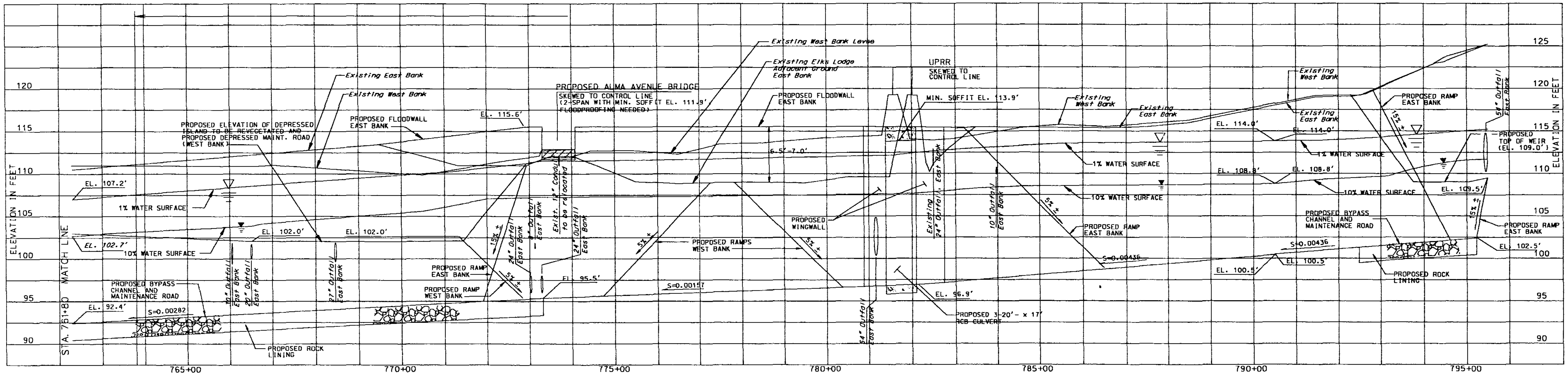
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-3



PROFILE

SCALE: 1"=100' H
1"=5' V



PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

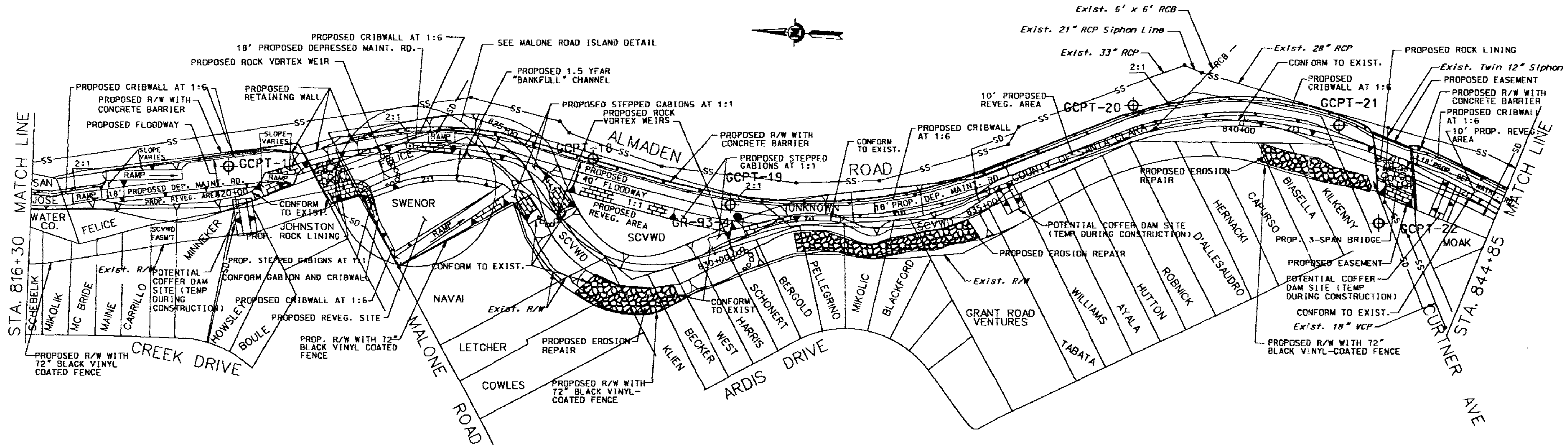
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
BYPASS CHANNEL PROFILE
STA 790+10 TO STA 795+00

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

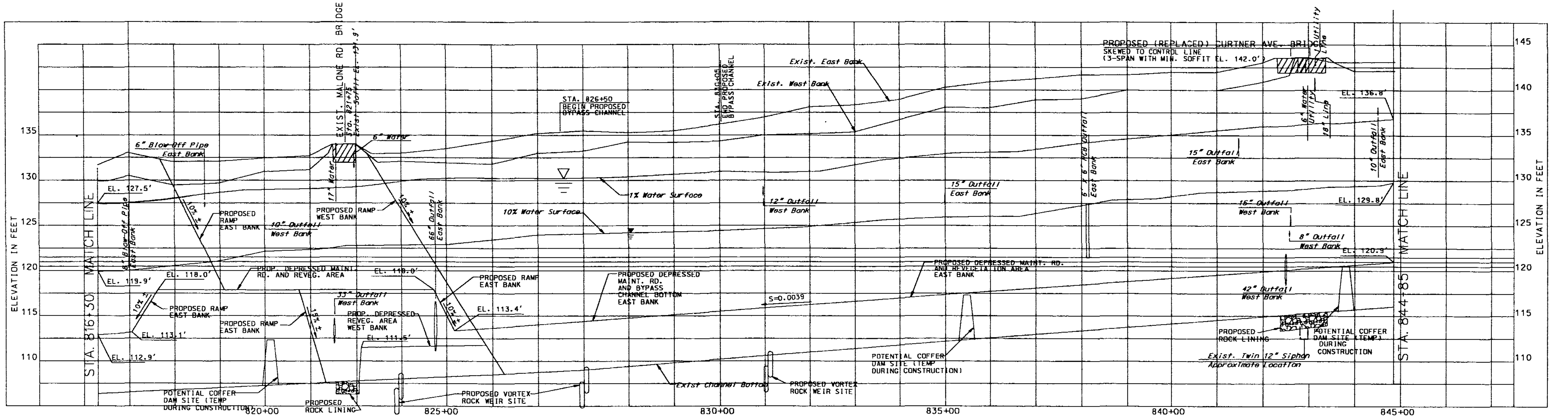
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-4



PLAN

SCALE: 1"=100'



PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

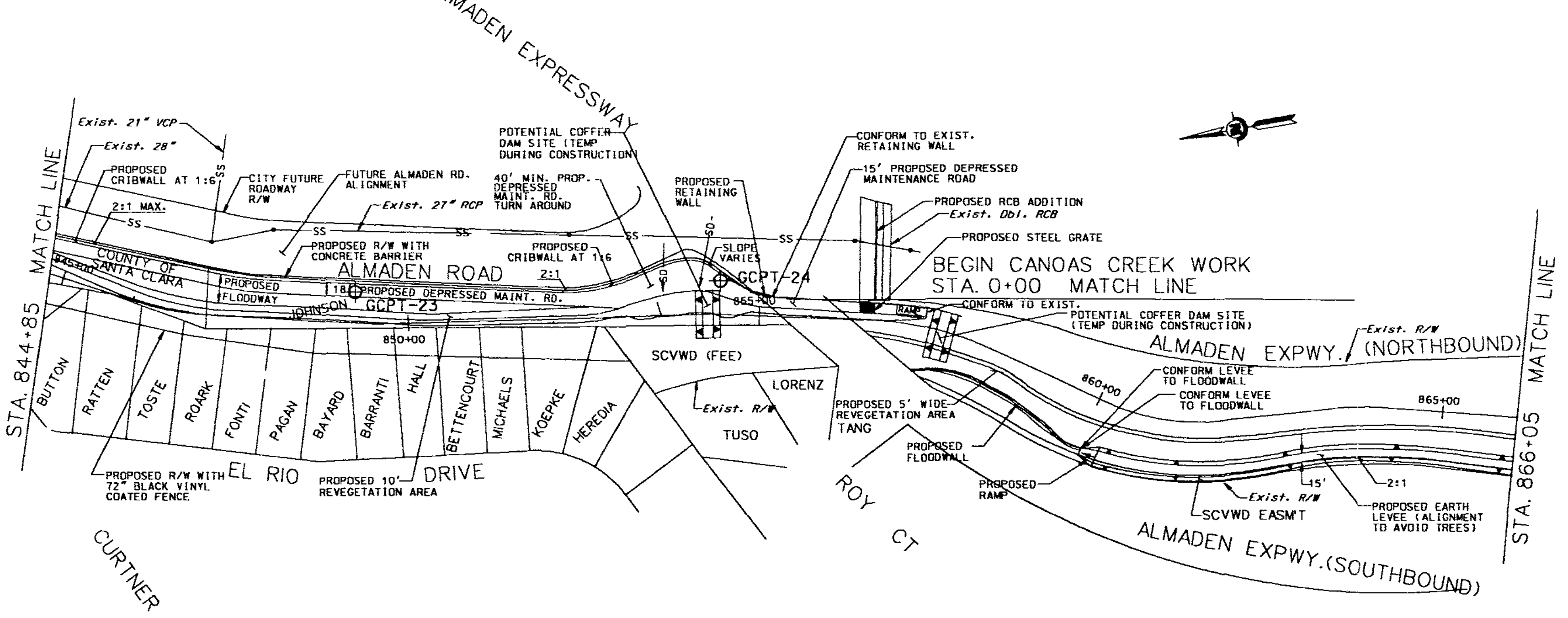
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 816+30 TO STA 844+85

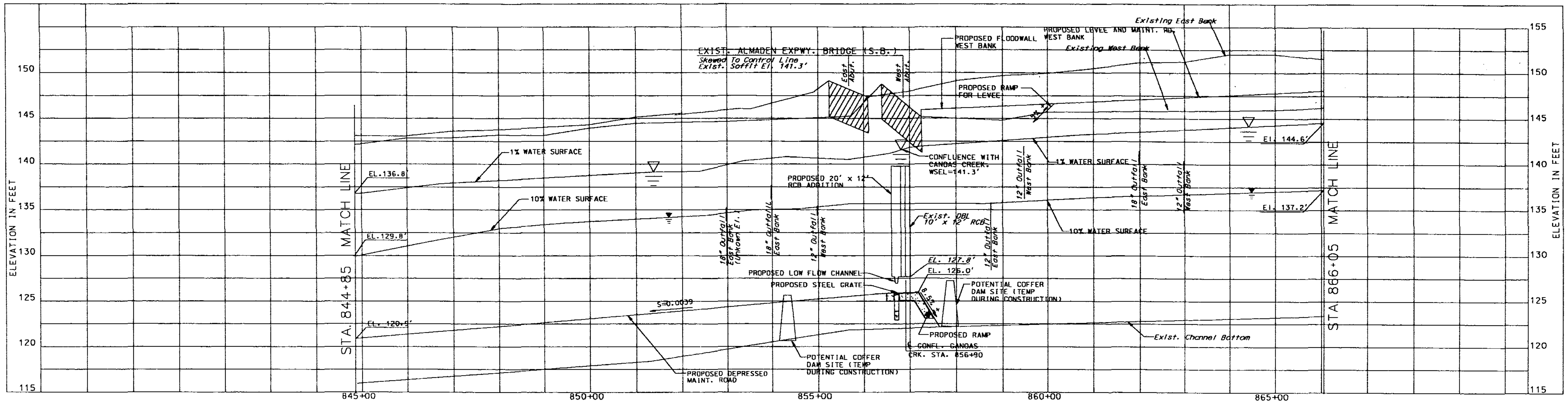
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-5



PLAN
SCALE: 1"=100'



PROFILE
SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

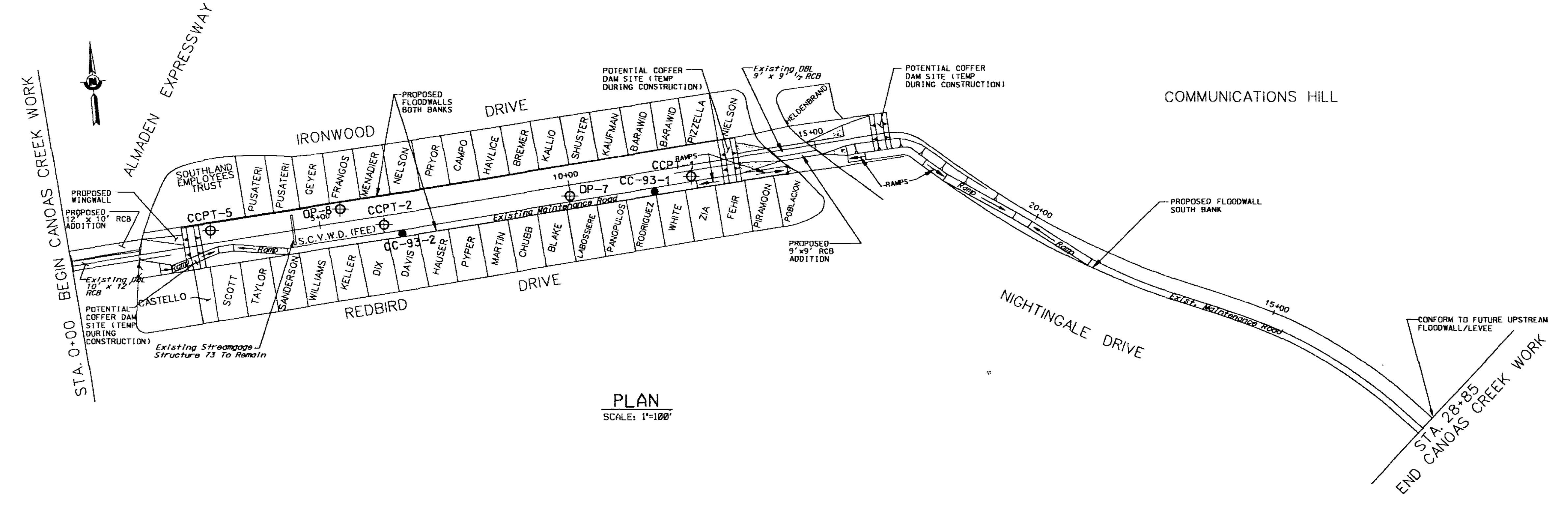
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 844+85 TO STA 866+05

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

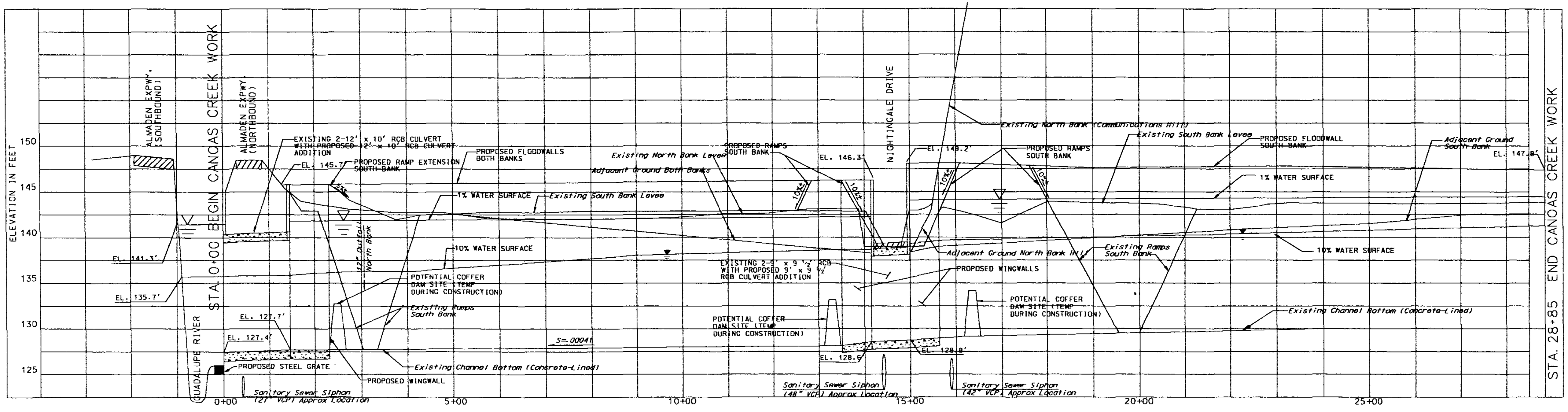
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-6


STA. 0+00 BEGIN CANOAS CREEK WORK
ALMADEN EXPRESSWAY
IRONWOOD DRIVE
REDBIRD DRIVE
COMMUNICATIONS HILL
NIGHTINGALE DRIVE
PROPOSED WINGWALL
PROPOSED 12' x 10' RCB ADDITION
Existing DBL 10' x 12' RCB
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
Existing Streamgage Structure 73 To Remain
PROPOSED FLOODWALLS BOTH BANKS
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
Existing DBL 9' x 9 1/2' RCB
PROPOSED 9'x9' RCB ADDITION
RAMPS
PROPOSED FLOODWALL SOUTH BANK
Exist. Maintenance Road
Existing Maintenance Road
CONFORM TO FUTURE UPSTREAM FLOODWALL/LEVEE
S.C.V.W.D. (FEE)
CCPT-5
OP-8
CCPT-2
CC-93-2
OP-7
CC-93-1
CCPT-1
5+00
10+00
15+00
20+00
Ramp
SOUTHLAND EMPLOYEES TRUST
PUSATERI
PUSATERI
GEYER
FRANGOS
MENADIER
NELSON
PRYOR
CAMPO
HAVLICE
BREMER
KALLIO
SHUSTER
KAUFMAN
BARAWID
BARAWID
PIZZELLA
NIELSON
HELDENBRAND
CASTELLO
SCOTT
TAYLOR
SANDERSON
WILLIAMS
KELLER
DIX
DAVIS
HAUSER
PYPER
MARTIN
CHUBB
BLAKE
LABOSSIERE
PANOPULOS
RODRIGUEZ
WHITE
ZIA
FEHR
PIRAMOON
POBLACION
STA. 28+85 END CANOAS CREEK WORK

PLAN

SCALE: 1"=100'


ELEVATION IN FEET
150
145
140
135
130
125
ALMADEN EXPWY. (SOUTHBOUND)
ALMADEN EXPWY. (NORTHBOUND)
STA. 0+00 BEGIN CANOAS CREEK WORK
GUADALUPE RIVER
EL. 141.3'
EL. 135.7'
EL. 127.7'
EL. 127.4'
EL. 145.7'
EXISTING 2-12' x 10' RCB CULVERT WITH PROPOSED 12' x 10' RCB CULVERT ADDITION
PROPOSED RAMP EXTENSION SOUTH BANK
PROPOSED FLOODWALLS BOTH BANKS
1% WATER SURFACE
Existing South Bank Levee
10% WATER SURFACE
12" Outfall North Bank
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
Existing Ramps South Bank
S=.0004
PROPOSED STEEL GRATE
PROPOSED WINGWALL
Existing Channel Bottom (Concrete-Lined)
Sanitary Sewer Siphon (27" VCP) Approx Location
Existing North Bank Levee
Adjacent Ground Both Banks
PROPOSED RAMPS SOUTH BANK
EL. 146.3
EXISTING 2-9' x 9 1/2' RCB WITH PROPOSED 9' x 9 1/2' RCB CULVERT ADDITION
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
EL. 128.6'
Sanitary Sewer Siphon (48" VCP) Approx Location
NIGHTINGALE DRIVE
EL. 148.2'
EL. 128.8'
Existing North Bank (Communications Hill)
PROPOSED RAMPS SOUTH BANK
10%
Adjacent Ground North Bank Hill
PROPOSED WINGWALLS
Existing Ramps South Bank
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
Sanitary Sewer Siphon (42" VCP) Approx Location
Existing South Bank Levee
PROPOSED FLOODWALL SOUTH BANK
1% WATER SURFACE
10% WATER SURFACE
Existing Channel Bottom (Concrete-Lined)
Adjacent Ground South Bank
EL. 147.8'
STA. 28+85 END CANOAS CREEK WORK
0+00
5+00
10+00
15+00
20+00
25+00

PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

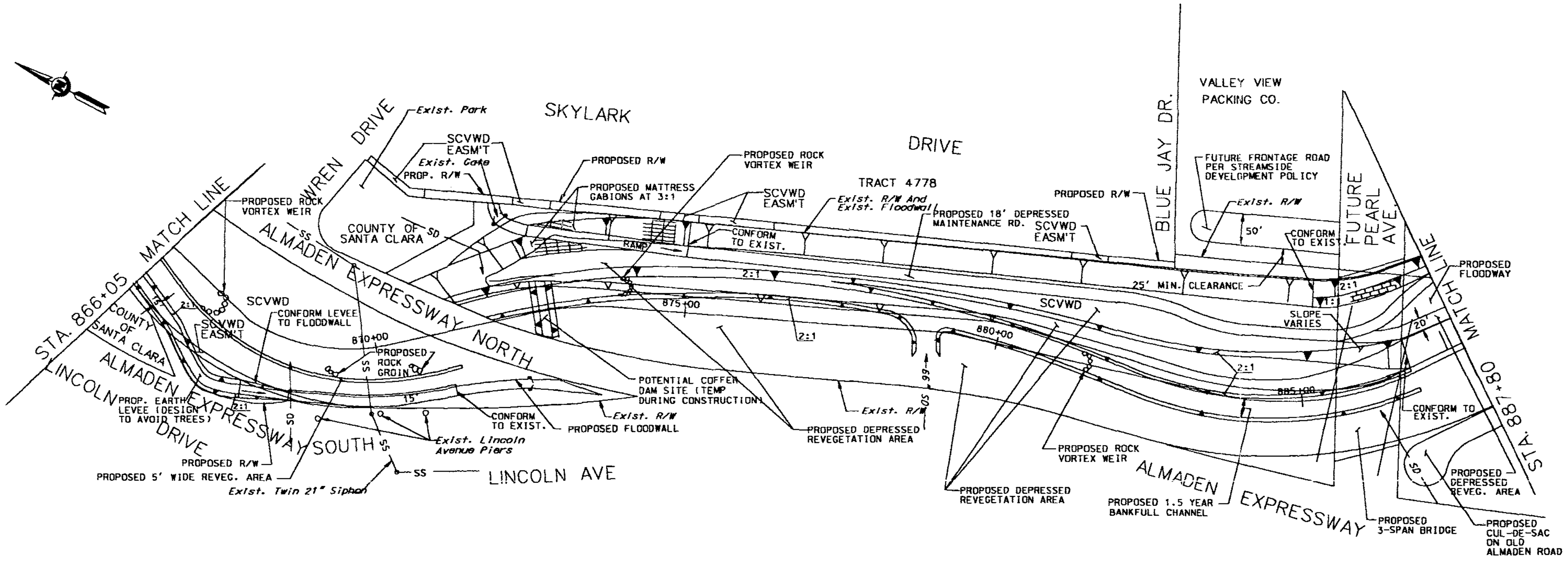
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
CANOAS CREEK
STA 0+00 TO STA 28+85

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

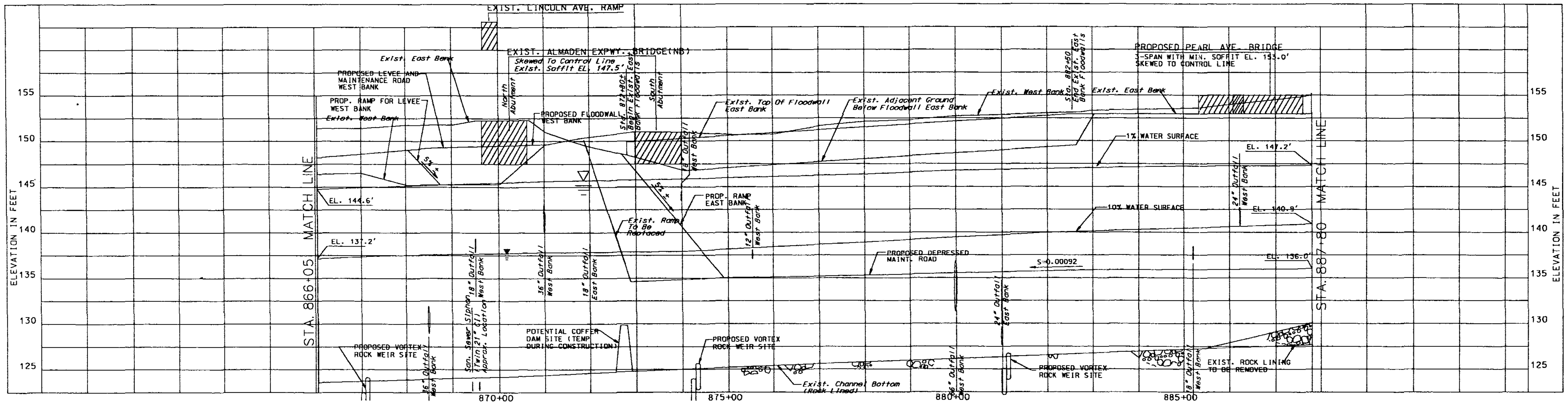
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-7



PLAN

SCALE: 1"=100'



EXIST. LINCOLN AVE. RAMP
EXIST. ALMADEN EXPWY. BRIDGE (NB)
Skewed To Control Line
Exist. Soffit EL. 147.5'
North Abutment
South Abutment
Sta. 872+80± Begin Exist. East Bank Floodwalls
Sta. 882+50 End Exist. East Bank Floodwalls
PROPOSED PEARL AVE. BRIDGE
3-SPAN WITH MIN. SOFFIT EL. 155.0'
SKEWED TO CONTROL LINE
Exist. East Bank
PROPOSED LEVEE AND MAINTENANCE ROAD WEST BANK
PROP. RAMP FOR LEVEE WEST BANK
Exist. West Bank
PROPOSED FLOODWALL WEST BANK
Exist. Top Of Floodwall East Bank
Exist. Adjacent Ground Below Floodwall East Bank
Exist. West Bank
Exist. East Bank
1% WATER SURFACE
EL. 147.2'
EL. 144.6'
10% WATER SURFACE
EL. 140.9'
EL. 137.2'
EL. 136.0'
PROP. RAMP EAST BANK
Exist. Ramp To Be Replaced
PROPOSED DEPRESSED MAINT. ROAD
S=0.00092
5%
18" Outfall West Bank
12" Outfall West Bank
24" Outfall West Bank
San. Sewer Siphon (Twin 21" SI) Approx. Location
18" Outfall West Bank
36" Outfall West Bank
18" Outfall East Bank
24" Outfall East Bank
36" Outfall West Bank
36" Outfall West Bank
18" Outfall West Bank
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
PROPOSED VORTEX ROCK WEIR SITE
PROPOSED VORTEX ROCK WEIR SITE
PROPOSED VORTEX ROCK WEIR SITE
Exist. Channel Bottom (Rock Lined)
EXIST. ROCK LINING TO BE REMOVED
STA. 866+05 MATCH LINE
STA. 887+80 MATCH LINE
ELEVATION IN FEET
155
150
145
140
135
130
125
870+00
875+00
880+00
885+00

PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

DRAWN BY:
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DESIGNED BY:
PROJECT COORDINATOR
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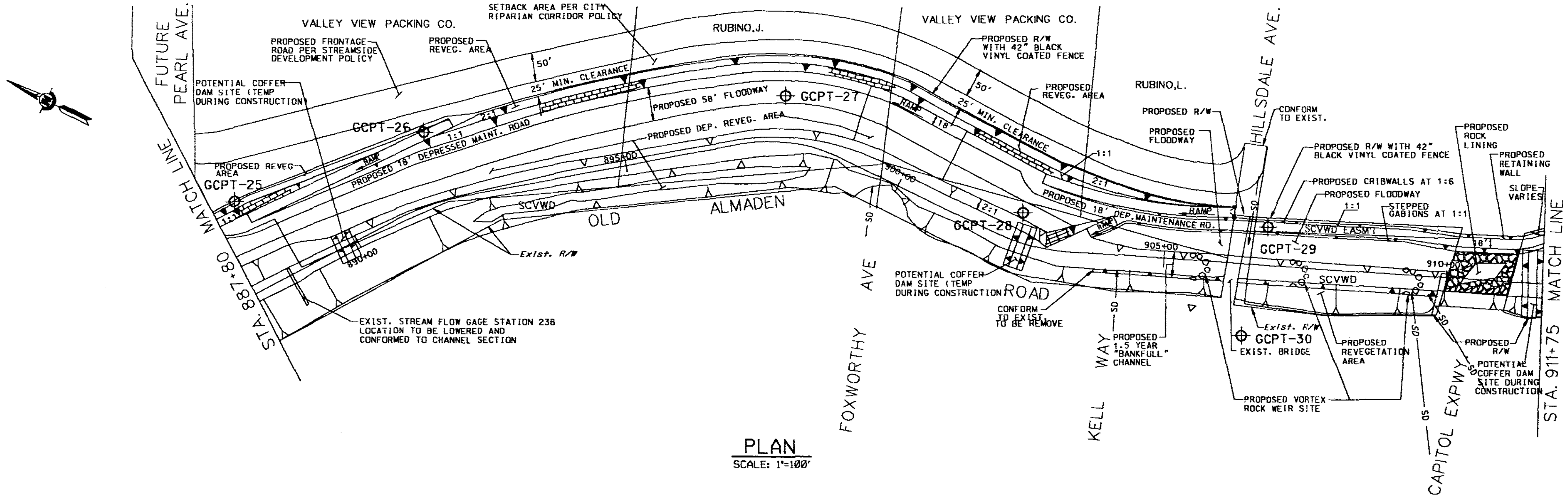
CHIEF, CIVIL DESIGN SECTION
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 866+05 TO STA 887+80

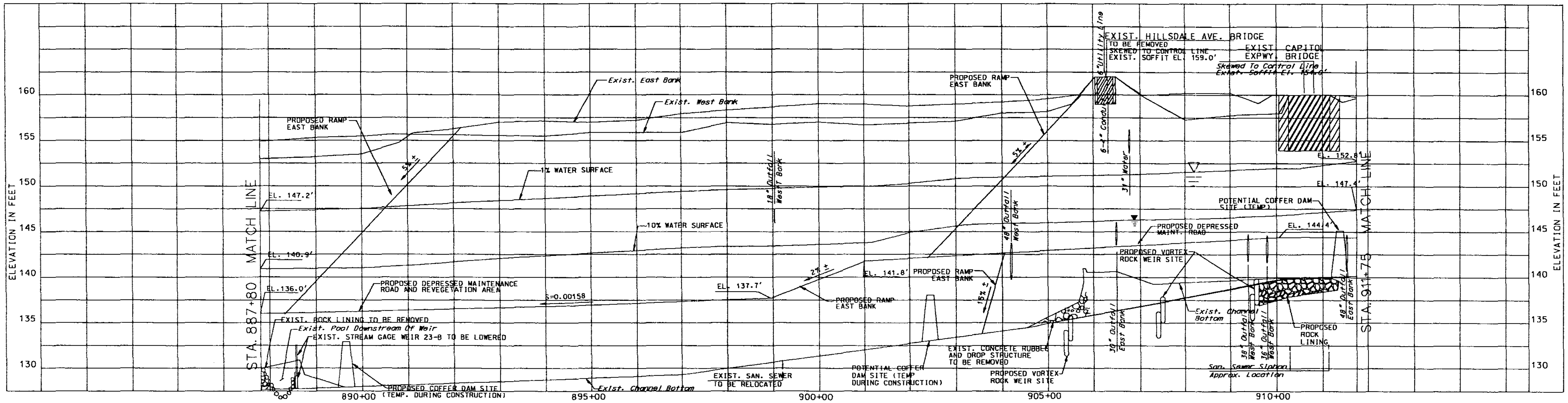
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-8


FUTURE PEARL AVE.
VALLEY VIEW PACKING CO.
SETBACK AREA PER CITY RIPARIAN CORRIDOR POLICY
RUBINO,J.
VALLEY VIEW PACKING CO.
PROPOSED FRONTAGE ROAD PER STREAMSIDE DEVELOPMENT POLICY
PROPOSED REVEG. AREA
50'
25' MIN. CLEARANCE
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
GCPT-26
PROPOSED 58' FLOODWAY
GCPT-27
PROPOSED R/W WITH 42" BLACK VINYL COATED FENCE
PROPOSED REVEG. AREA
RUBINO,L.
HILLSDALE AVE.
PROPOSED R/W
PROPOSED FLOODWAY
CONFORM TO EXIST.
2:1
1:1
RAMP
PROPOSED 18' DEPRESSED MAINT. ROAD
PROPOSED DEP. REVEG. AREA
895+00
MATCH LINE
PROPOSED REVEG AREA
GCPT-25
SCVWD
OLD
ALMADEN
18'
25' MIN. CLEARANCE
900+00
PROPOSED 18' DEP. MAINTENANCE RD.
PROPOSED R/W WITH 42" BLACK VINYL COATED FENCE
PROPOSED CRIBWALLS AT 1:6
PROPOSED FLOODWAY
STEPPED GABIONS AT 1:1
SCVWD EASM'T
PROPOSED ROCK LINING
PROPOSED RETAINING WALL
SLOPE VARIES
18'
GCPT-28
GCPT-29
905+00
910+00
SCVWD
890+00
Exist. R/W
STA. 887+80
EXIST. STREAM FLOW GAGE STATION 23B LOCATION TO BE LOWERED AND CONFORMED TO CHANNEL SECTION
FOXWORTHY AVE
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
ROAD
CONFORM TO EXIST. TO BE REMOVE
KELL WAY
PROPOSED 1.5 YEAR "BANKFULL" CHANNEL
Exist. R/W
GCPT-30
EXIST. BRIDGE
PROPOSED REVEGETATION AREA
PROPOSED VORTEX ROCK WEIR SITE
CAPITOL EXPWY
PROPOSED R/W
POTENTIAL COFFER DAM SITE DURING CONSTRUCTION
MATCH LINE
STA. 911+75
SD

PLAN

SCALE: 1"=100'



PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

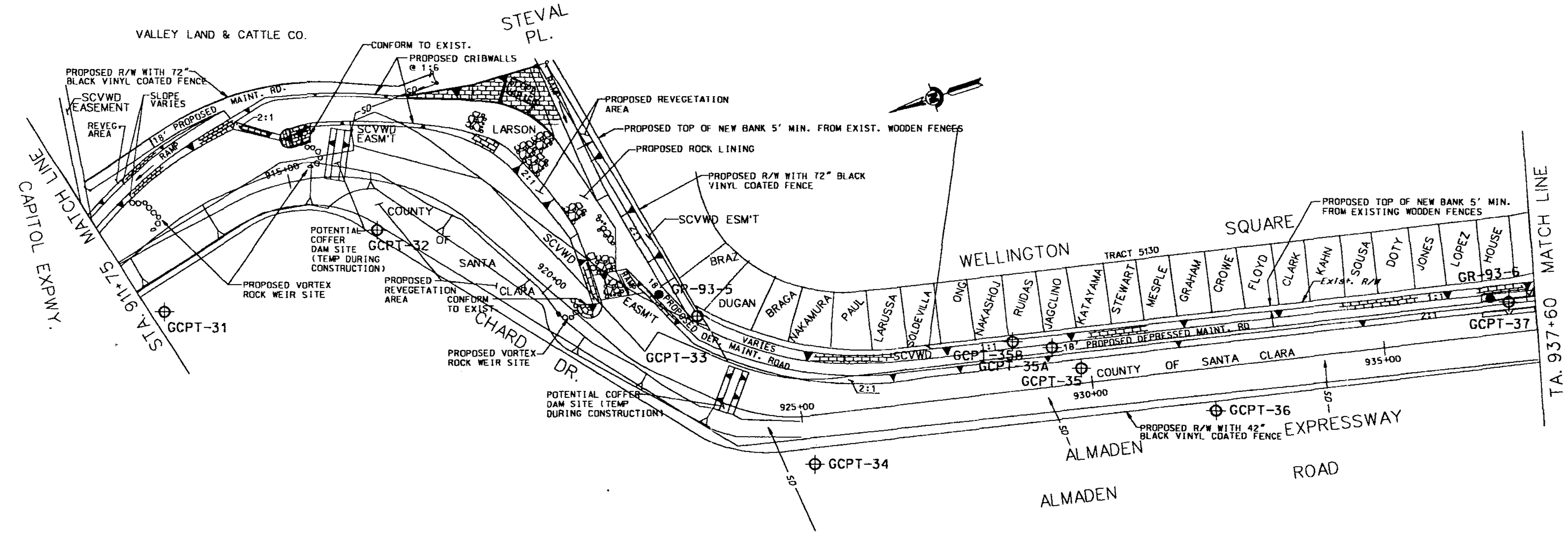
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 887+80 TO STA 911+75

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
DATE:
CHIEF, PLANNING/ENGINEERING DIVISION

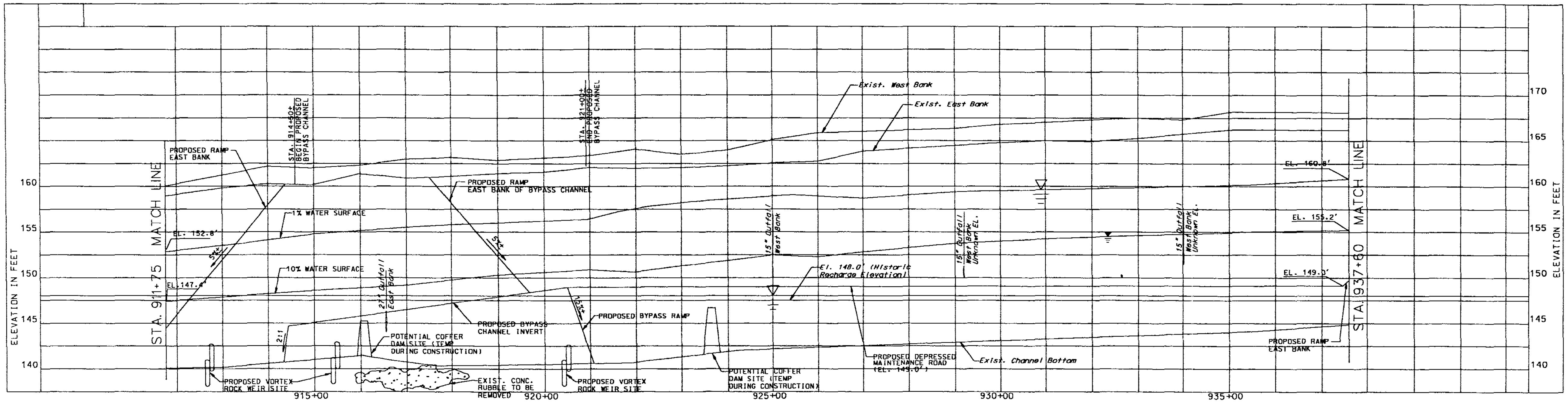
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-9



PLAN

SCALE: 1"=100'



PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

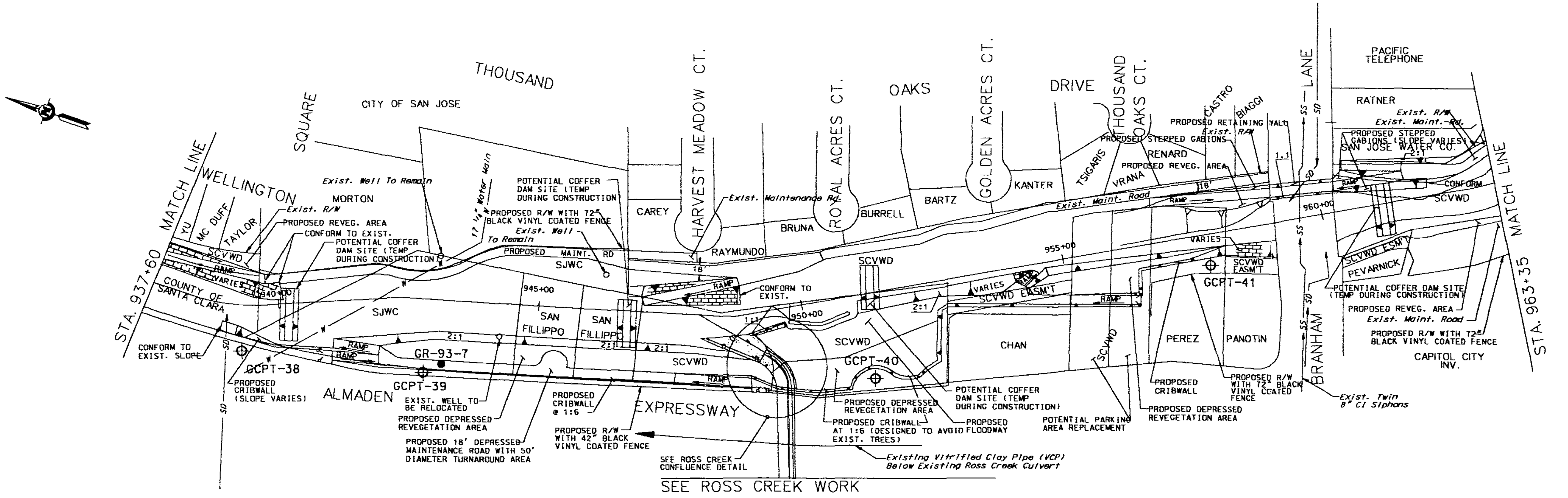
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 911+75 TO STA 937+60

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

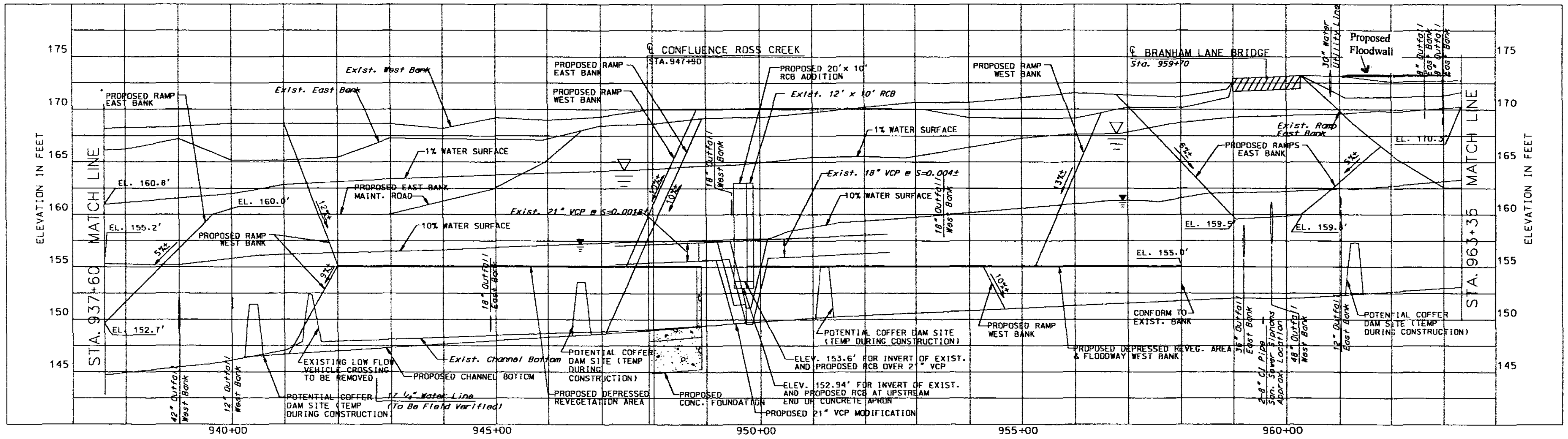
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-10



PLAN

SCALE: 1"=100'


175
170
165
160
155
150
145
ELEVATION IN FEET
MATCH LINE
STA. 937+60
MATCH LINE
STA. 963+35
CONFLUENCE ROSS CREEK STA. 947+90
BRANHAM LANE BRIDGE Sta. 959+70
Proposed Floodwall
30" Water Utility Line
Exist. West Bank
Exist. East Bank
PROPOSED RAMP EAST BANK
PROPOSED RAMP WEST BANK
PROPOSED 20' x 10' RCB ADDITION
Exist. 12' x 10' RCB
1% WATER SURFACE
10% WATER SURFACE
PROPOSED EAST BANK MAINT. ROAD
Exist. 21" VCP @ S=0.0018±
Exist. 18" VCP @ S=0.004±
EL. 160.8'
EL. 160.0'
EL. 155.2'
EL. 152.7'
EL. 159.5'
EL. 159.6'
EL. 155.0'
EL. 170.3'
Exist. Ramp East Bank
PROPOSED RAMPS EAST BANK
CONFORM TO EXIST. BANK
PROPOSED RAMP WEST BANK
EXISTING LOW FLOW VEHICLE CROSSING TO BE REMOVED
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
17 1/4" Water Line (To Be Field Verified)
PROPOSED CHANNEL BOTTOM
Exist. Channel Bottom
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
PROPOSED DEPRESSED REVEGETATION AREA
PROPOSED CONC. FOUNDATION
PROPOSED 21" VCP MODIFICATION
ELEV. 153.6' FOR INVERT OF EXIST. AND PROPOSED RCB OVER 21" VCP
ELEV. 152.94' FOR INVERT OF EXIST. AND PROPOSED RCB AT UPSTREAM END OF CONCRETE APRON
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
PROPOSED DEPRESSED REVEG. AREA & FLOODWAY WEST BANK
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
42" Outfall West Bank
12" Outfall West Bank
18" Outfall East Bank
18" Outfall West Bank
18" Outfall West Bank
36" Outfall East Bank
2-8" CI Pipe San. Sewer Siphons Approx. Location
48" Outfall West Bank
12" Outfall East Bank
8" Outfall East Bank
8" Outfall East Bank
5%±
12%±
10%±
3%±
13%±
940+00
945+00
950+00
955+00
960+00

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

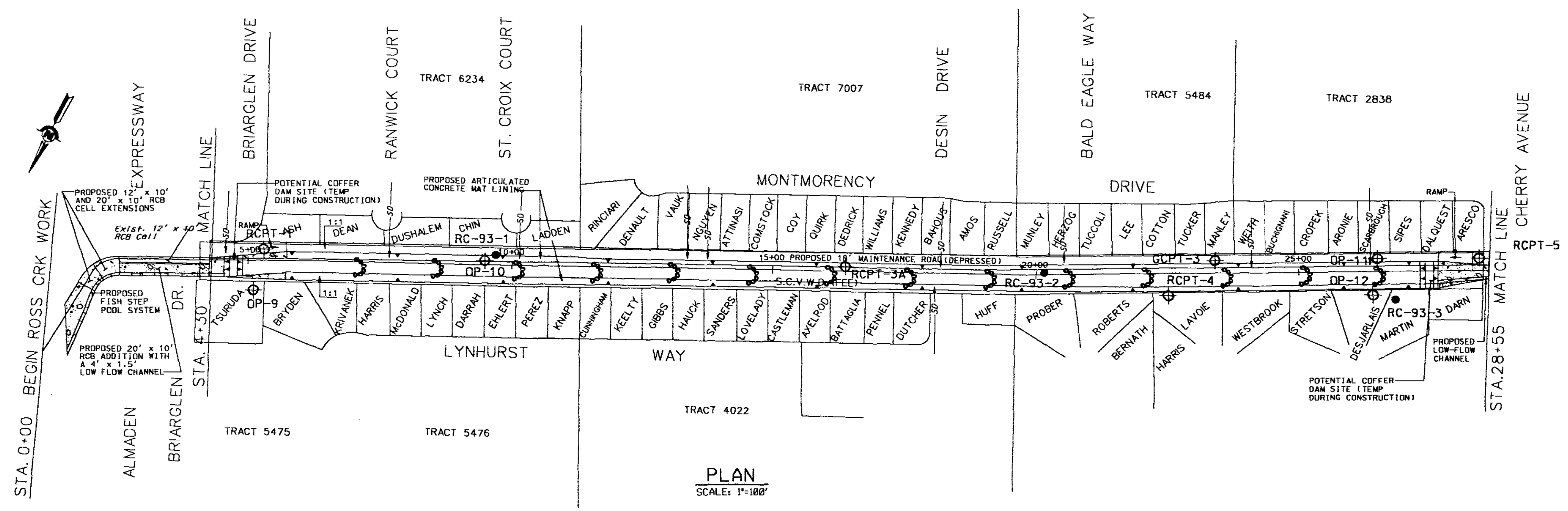
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 937+60 TO STA 963+35

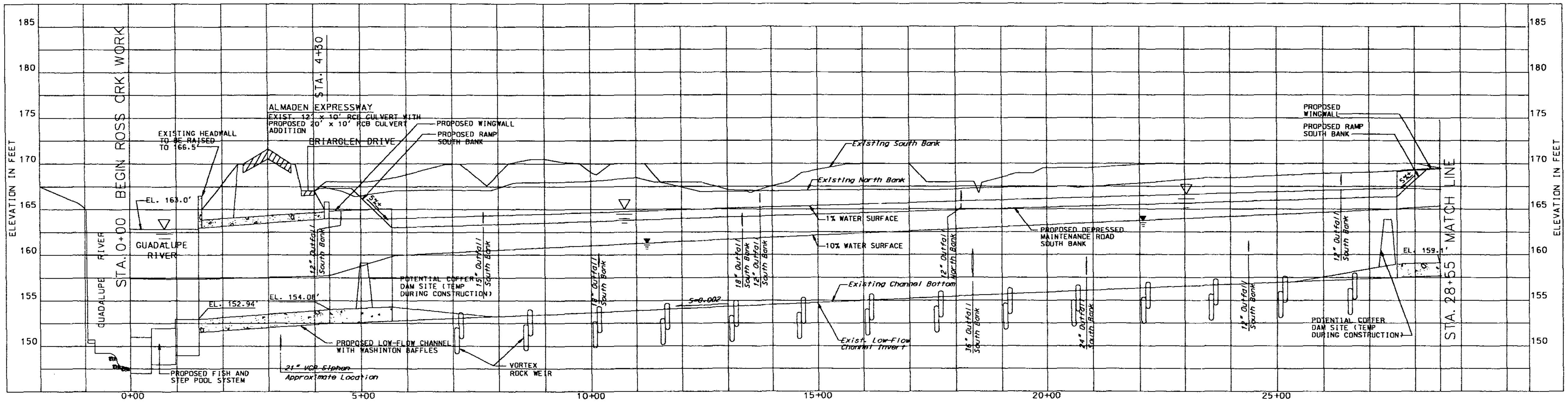
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-11



PLAN
SCALE: 1"=100'



PROFILE
SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

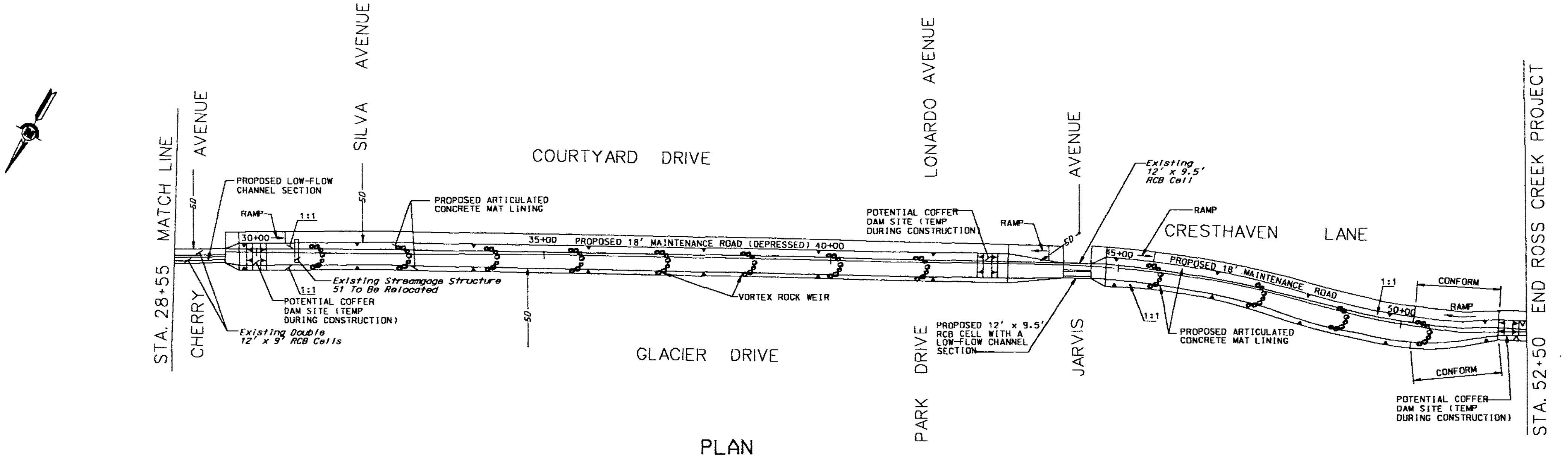
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
ROSS CREEK
STA 0+00 TO STA 28+55

APPROVED: DATE:
CHIEF, PLANNING/ENGINEERING DIVISION

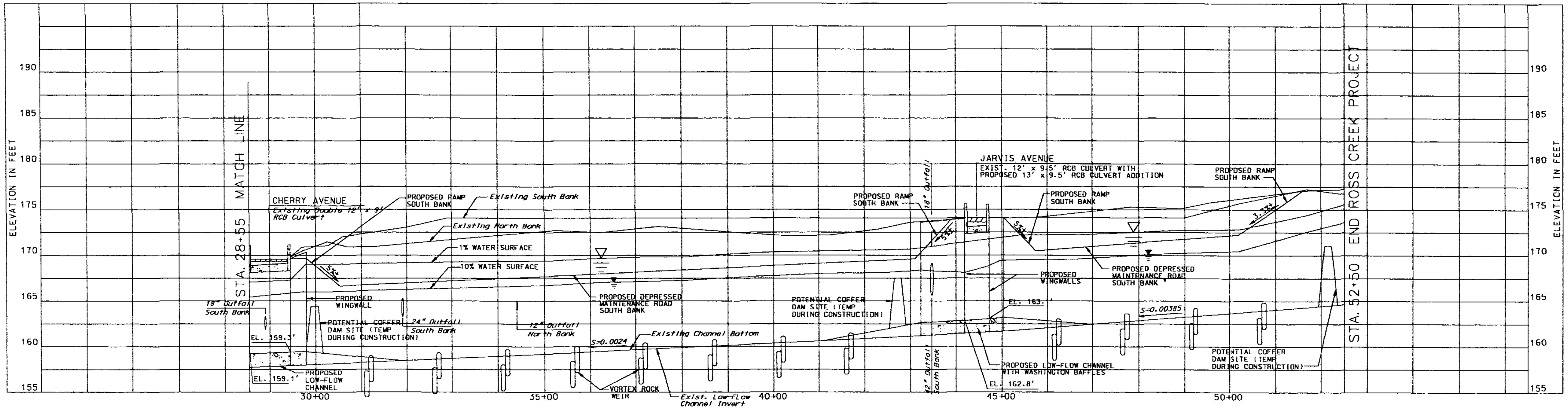
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-12


STA. 28+55 MATCH LINE
CHERRY AVENUE
SILVA AVENUE
COURTYARD DRIVE
GLACIER DRIVE
LONARDO AVENUE
ROSS PARK DRIVE
JARVIS AVENUE
CRESTHAVEN LANE
STA. 52+50 END ROSS CREEK PROJECT
PROPOSED LOW-FLOW CHANNEL SECTION
RAMP
1:1
30+00
35+00
40+00
45+00
50+00
SD
PROPOSED ARTICULATED CONCRETE MAT LINING
PROPOSED 18' MAINTENANCE ROAD (DEPRESSED)
Existing Streamgage Structure 51 To Be Relocated
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
Existing Double 12' x 9' RCB Cells
VORTEX ROCK WEIR
Existing 12' x 9.5' RCB Cell
PROPOSED 12' x 9.5' RCB CELL WITH A LOW-FLOW CHANNEL SECTION
PROPOSED 18' MAINTENANCE ROAD
CONFORM

PLAN

SCALE: 1"=100'



PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

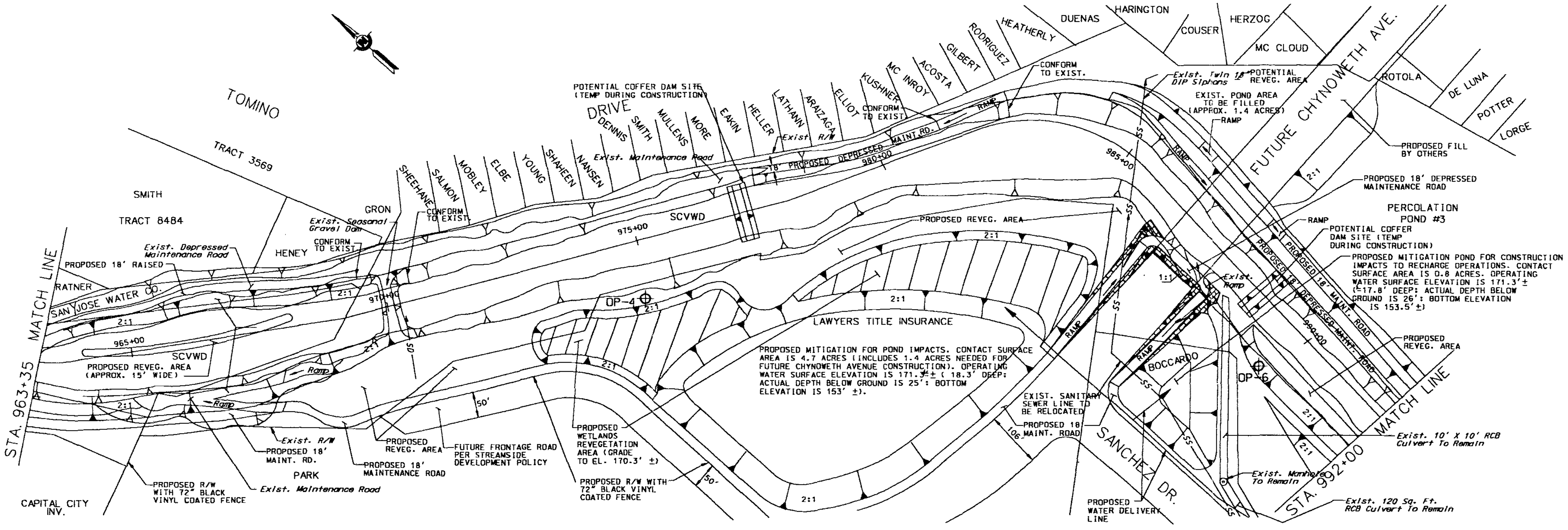
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
ROSS CREEK
STA 28+55 TO STA 52+50

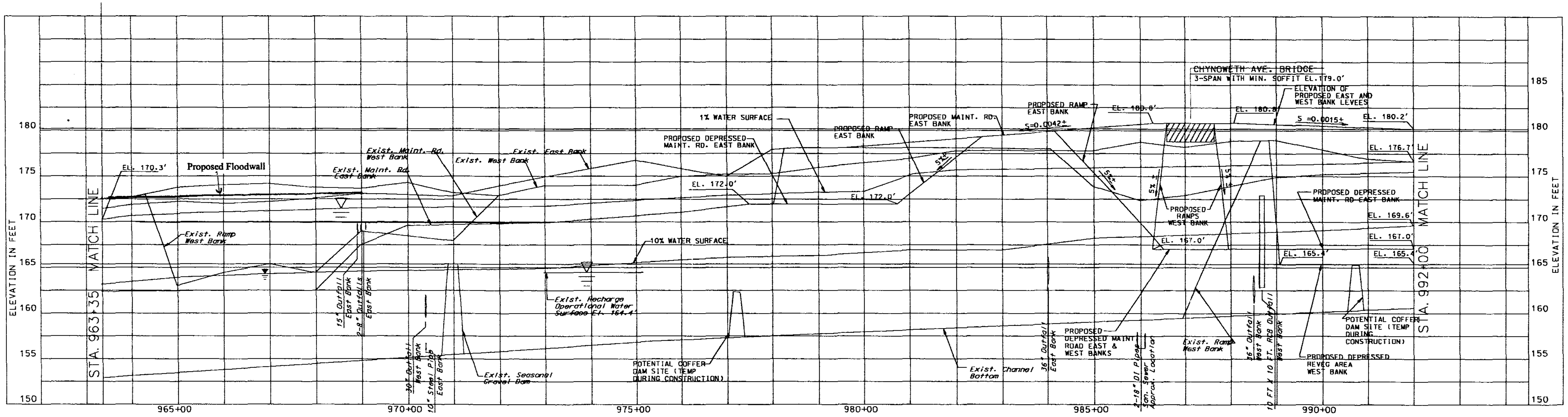
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-13


TOMINO
TRACT 3569
SMITH
TRACT 8484
GRON
HENEY
RATNER
SAN JOSE WATER CO.
STA. 963+35 MATCH LINE
PROPOSED 18' RAISED
Exist. Depressed Maintenance Road
Exist. Seasonal Gravel Dam
CONFORM TO EXIST.
SHEEHANE
SALMON
MOBLEY
ELBE
YOUNG
SHAHEEN
NANSEN
DENNIS
SMITH
MULLENS
MORE
EAKIN
HELLER
LATHANN
ARAIZAGA
ELLIOT
KUSHNER
MC INROY
ACOSTA
GILBERT
RODRIGUEZ
HEATHERLY
DUENAS
HARINGTON
COUSER
HERZOG
MC CLOUD
ROTOLA
DE LUNA
POTTER
LORGE
DRIVE
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
Exist. Maintenance Road
Exist. R/W
18' PROPOSED DEPRESSED MAINT. RD.
CONFORM TO EXIST.
Exist. Twin 18" DIP Siphons
POTENTIAL REVEG. AREA
EXIST. POND AREA TO BE FILLED (APPROX. 1.4 ACRES)
RAMP
FUTURE CHYNOWETH AVE.
PROPOSED FILL BY OTHERS
PROPOSED 18' DEPRESSED MAINTENANCE ROAD
PERCOLATION POND #3
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
PROPOSED MITIGATION POND FOR CONSTRUCTION IMPACTS TO RECHARGE OPERATIONS. CONTACT SURFACE AREA IS 0.8 ACRES. OPERATING WATER SURFACE ELEVATION IS 171.3' ± (±17.8' DEEP; ACTUAL DEPTH BELOW GROUND IS 26'; BOTTOM ELEVATION IS 153.5' ±)
PROPOSED REVEG. AREA
SCVWD
975+00
980+00
985+00
990+00
970+00
965+00
2:1
1:1
Ramp
OP-4
OP-6
LAWYERS TITLE INSURANCE
PROPOSED MITIGATION FOR POND IMPACTS. CONTACT SURFACE AREA IS 4.7 ACRES (INCLUDES 1.4 ACRES NEEDED FOR FUTURE CHYNOWETH AVENUE CONSTRUCTION). OPERATING WATER SURFACE ELEVATION IS 171.3'± (18.3' DEEP; ACTUAL DEPTH BELOW GROUND IS 25'; BOTTOM ELEVATION IS 153' ±).
EXIST. SANITARY SEWER LINE TO BE RELOCATED
PROPOSED 18' MAINT. ROAD
BOCCARDO
SANCHEZ DR.
SS
Exist. Ramp
MATCH LINE
STA. 992+00
Exist. 10' X 10' RCB Culvert To Remain
Exist. Manhole To Remain
Exist. 120 Sq. Ft. RCB Culvert To Remain
PROPOSED WATER DELIVERY LINE
PROPOSED REVEG. AREA (APPROX. 15' WIDE)
Exist. R/W
PROPOSED 18' MAINT. RD.
PROPOSED REVEG. AREA
FUTURE FRONTAGE ROAD PER STREAMSIDE DEVELOPMENT POLICY
PROPOSED 18' MAINTENANCE ROAD
PARK
PROPOSED R/W WITH 72" BLACK VINYL COATED FENCE
Exist. Maintenance Road
CAPITAL CITY INV.
PROPOSED WETLANDS REVEGETATION AREA (GRADE TO EL. 170.3' ±)
PROPOSED R/W WITH 72" BLACK VINYL COATED FENCE
50'
106'

PLAN
SCALE: 1"=100'



PROFILE
SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

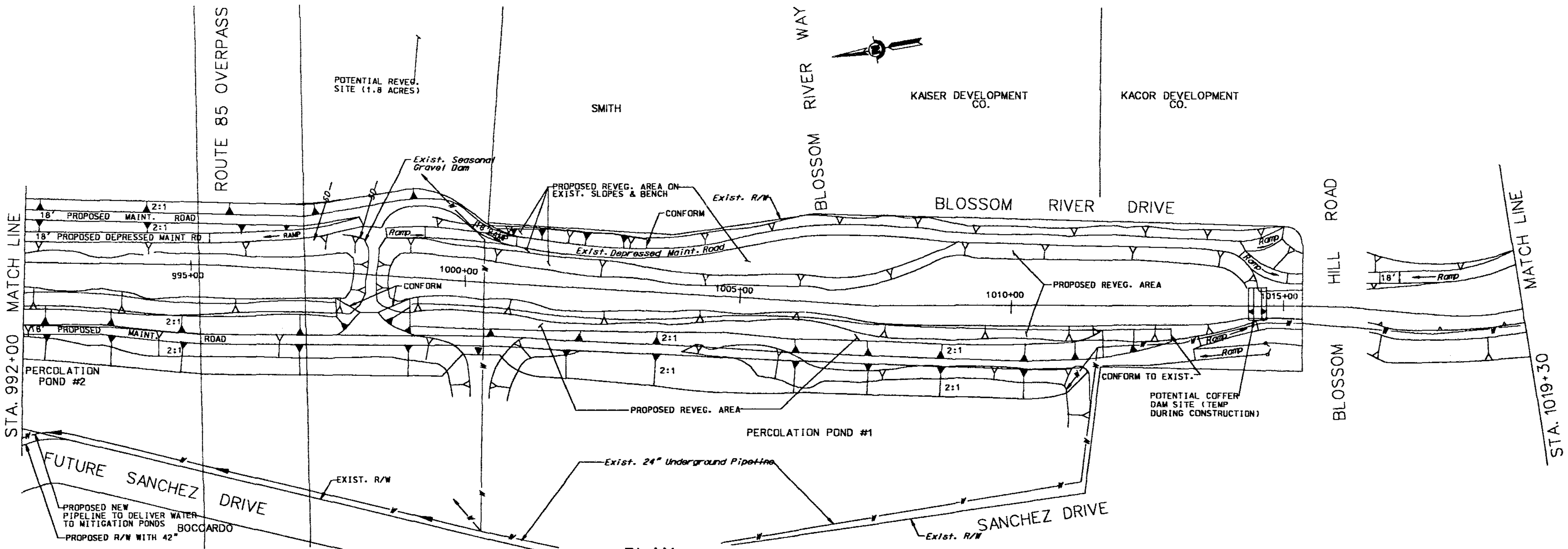
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 963+35 TO STA 992+00

APPROVAL RECOMMENDED: CHIEF, ENGINEERING BRANCH
APPROVED: CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

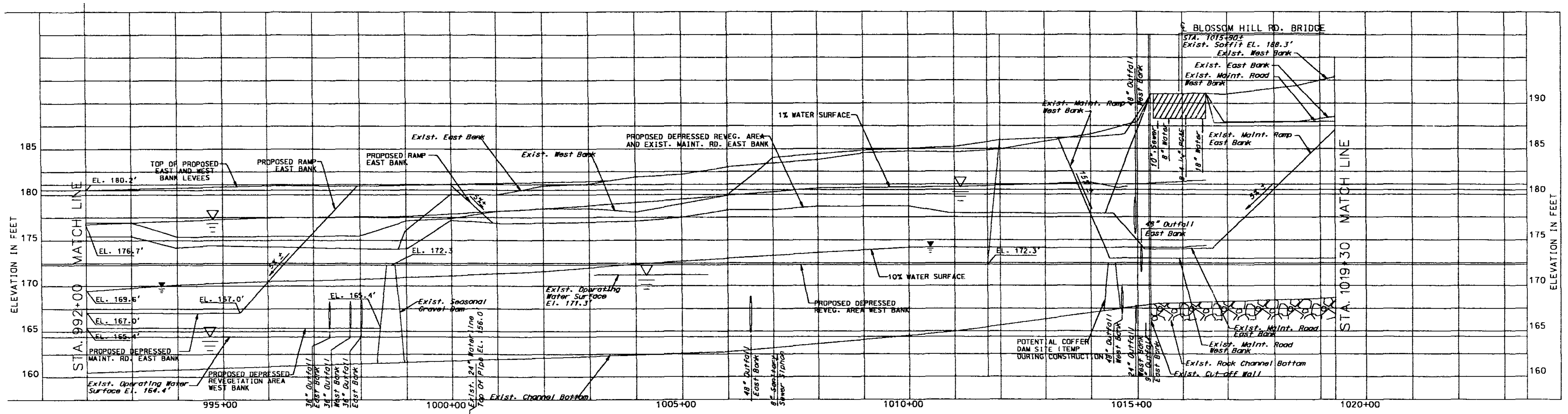
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-14



PLAN

SCALE: 1"=100'



PROFILE

SCALE: 1"=100' H
1"=5' V

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

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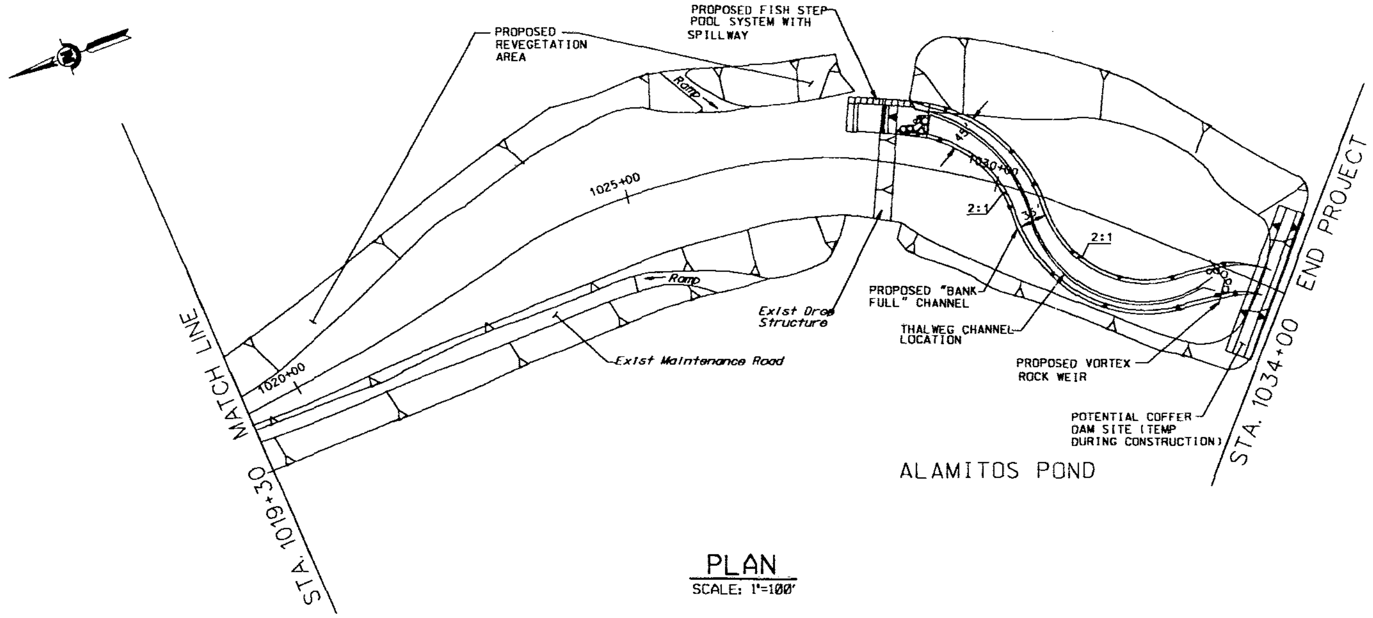
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 992+00 TO STA 1019+30

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

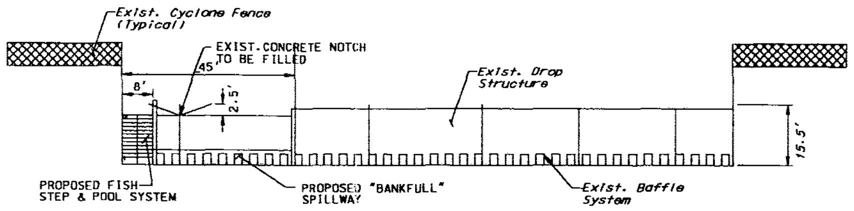
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-15

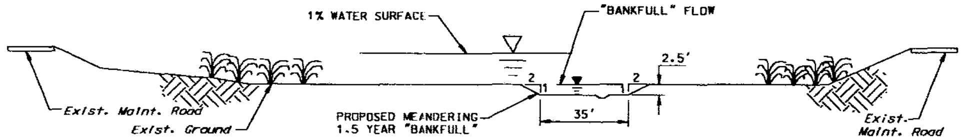

PROPOSED REVEGETATION AREA
PROPOSED FISH STEP POOL SYSTEM WITH SPILLWAY
Ramp
1025+00
1020+00
1030+00
2:1
2:1
Ramp
Exist Drop Structure
Exist Maintenance Road
PROPOSED "BANKFULL" CHANNEL
THALWEG CHANNEL LOCATION
PROPOSED VORTEX ROCK WEIR
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
ALAMITOS POND
STA. 1019+30 MATCH LINE
STA. 1034+00 END PROJECT

PLAN

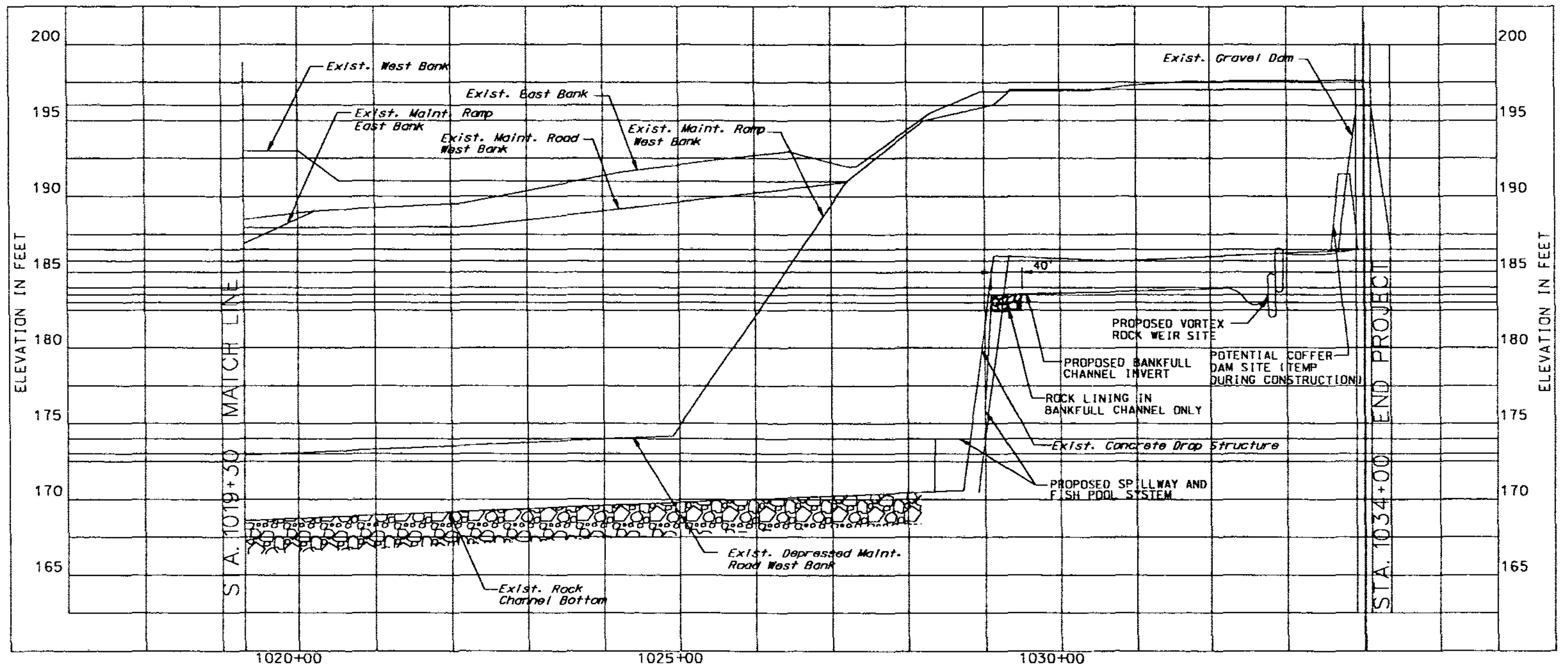
SCALE: 1"=100'


Exist. Cyclone Fence (Typical)
EXIST. CONCRETE NOTCH TO BE FILLED
45
8'
2.5'
Exist. Drop Structure
15.5'
PROPOSED FISH STEP & POOL SYSTEM
PROPOSED "BANKFULL" SPILLWAY
Exist. Baffle System

EXISTING DROP STRUCTURE WITH PROPOSED FISH STEP POOL SYSTEM
STA. 1028+35± TO STA. 1029+15±
NTS



TYPICAL SECTION
STA. 1029+15± TO STA. 1034+00±
NTS


200
195
190
185
180
175
170
165
ELEVATION IN FEET
Exist. West Bank
Exist. East Bank
Exist. Maint. Ramp East Bank
Exist. Maint. Road West Bank
Exist. Maint. Ramp West Bank
Exist. Gravel Dam
40'
PROPOSED VORTEX ROCK WEIR SITE
PROPOSED BANKFULL CHANNEL INVERT
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
ROCK LINING IN BANKFULL CHANNEL ONLY
Exist. Concrete Drop Structure
PROPOSED SPILLWAY AND FISH POOL SYSTEM
Exist. Depressed Maint. Road West Bank
Exist. Rock Channel Bottom
STA. 1019+30 MATCH LINE
STA. 1034+00 END PROJECT
1020+00
1025+00
1030+00

PROFILE

SCALE: 1"=100' H
1"=5' V

NOTE:

ONLY REVEGETATION FEATURES INCLUDED IN FEDERAL PROJECT. ALL OTHER FEATURES TO BE LOCAL RESPONSIBILITY.

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

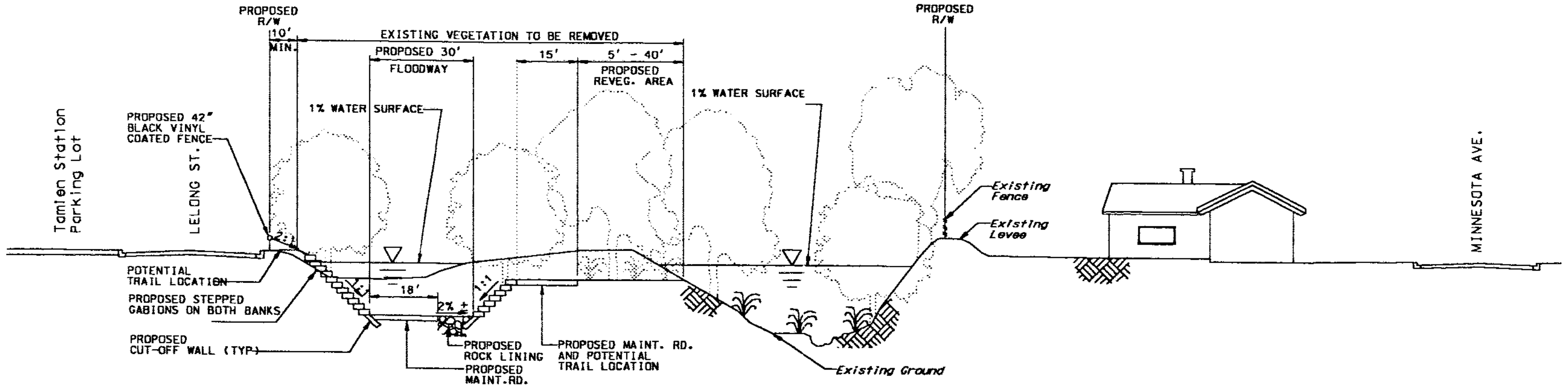
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN AND PROFILE
STA 1019+30 TO STA 1034+00

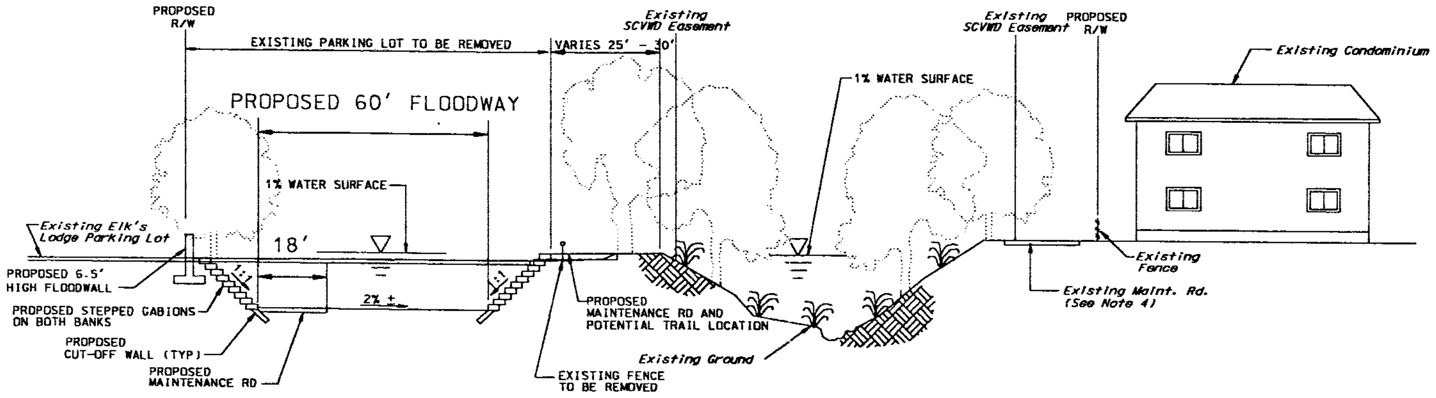
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-16



TYPICAL SECTION
STA. 751+85± TO STA. 769+00±
NTS


PROPOSED R/W
EXISTING PARKING LOT TO BE REMOVED
VARIES 25' - 30'
Existing SCVWD Easement
Existing SCVWD Easement
PROPOSED R/W
Existing Condominium
PROPOSED 60' FLOODWAY
1% WATER SURFACE
1% WATER SURFACE
Existing Elk's Lodge Parking Lot
18'
PROPOSED 6.5' HIGH FLOODWALL
1:1
PROPOSED STEPPED GABIONS ON BOTH BANKS
2% ±
1:1
PROPOSED CUT-OFF WALL (TYP)
PROPOSED MAINTENANCE RD
PROPOSED MAINTENANCE RD AND POTENTIAL TRAIL LOCATION
EXISTING FENCE TO BE REMOVED
Existing Ground
Existing Fence
Existing Maint. Rd. (See Note 4)

TYPICAL SECTION
STA. 775+70± TO STA. 780+70±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

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DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 751+85 TO STA 780+70

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

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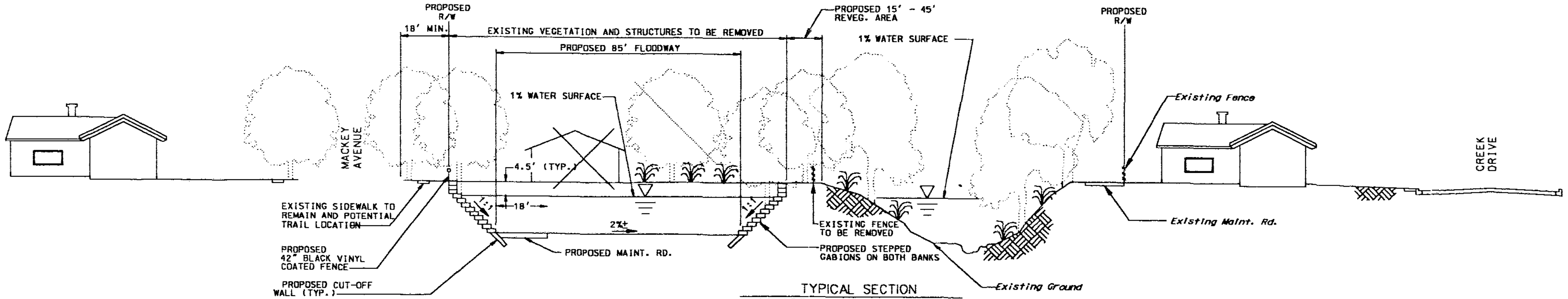
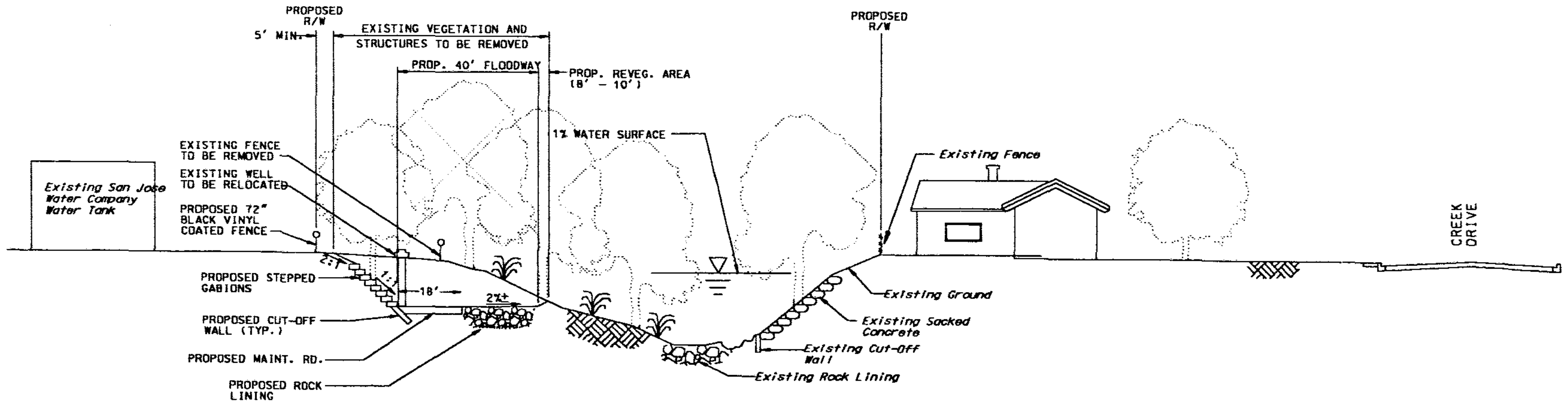
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

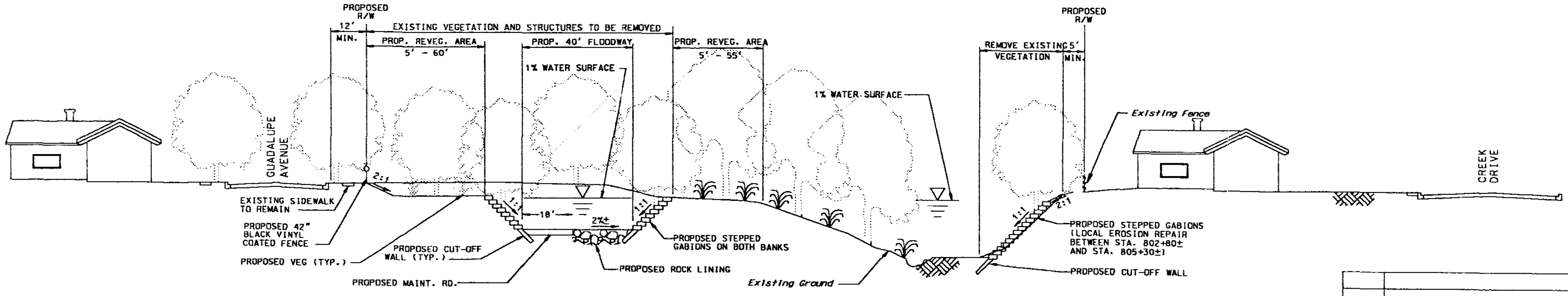
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-17


PROPOSED R/W
18' MIN.
EXISTING VEGETATION AND STRUCTURES TO BE REMOVED
PROPOSED 85' FLOODWAY
PROPOSED 15' - 45' REVEG. AREA
1% WATER SURFACE
1% WATER SURFACE
PROPOSED R/W
Existing Fence
MACKEY AVENUE
4.5' (TYP.)
18'
1:1
1:1
2%±
CREEK DRIVE
EXISTING SIDEWALK TO REMAIN AND POTENTIAL TRAIL LOCATION
PROPOSED 42" BLACK VINYL COATED FENCE
PROPOSED CUT-OFF WALL (TYP.)
PROPOSED MAINT. RD.
EXISTING FENCE TO BE REMOVED
PROPOSED STEPPED GABIONS ON BOTH BANKS
Existing Maint. Rd.
Existing Ground

TYPICAL SECTION
STA. 785+25± TO STA. 792+80±
NTS



TYPICAL SECTION
STA. 797+00± TO STA. 801+00±
NTS


PROPOSED R/W
12' MIN.
EXISTING VEGETATION AND STRUCTURES TO BE REMOVED
PROP. REVEG. AREA 5' - 60'
PROP. 40' FLOODWAY
PROP. REVEG. AREA 5' - 55'
1% WATER SURFACE
1% WATER SURFACE
REMOVE EXISTING VEGETATION
5' MIN.
PROPOSED R/W
Existing Fence
GUADALUPE AVENUE
2:1
1:1
1:1
18'
2%±
1:1
2:1
CREEK DRIVE
EXISTING SIDEWALK TO REMAIN
PROPOSED 42" BLACK VINYL COATED FENCE
PROPOSED VEG (TYP.)
PROPOSED CUT-OFF WALL (TYP.)
PROPOSED MAINT. RD.
PROPOSED ROCK LINING
PROPOSED STEPPED GABIONS ON BOTH BANKS
Existing Ground
PROPOSED STEPPED GABIONS (LOCAL EROSION REPAIR BETWEEN STA. 802+80± AND STA. 805+30±)
PROPOSED CUT-OFF WALL

TYPICAL SECTION
STA. 802+00± TO STA. 806+00±

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

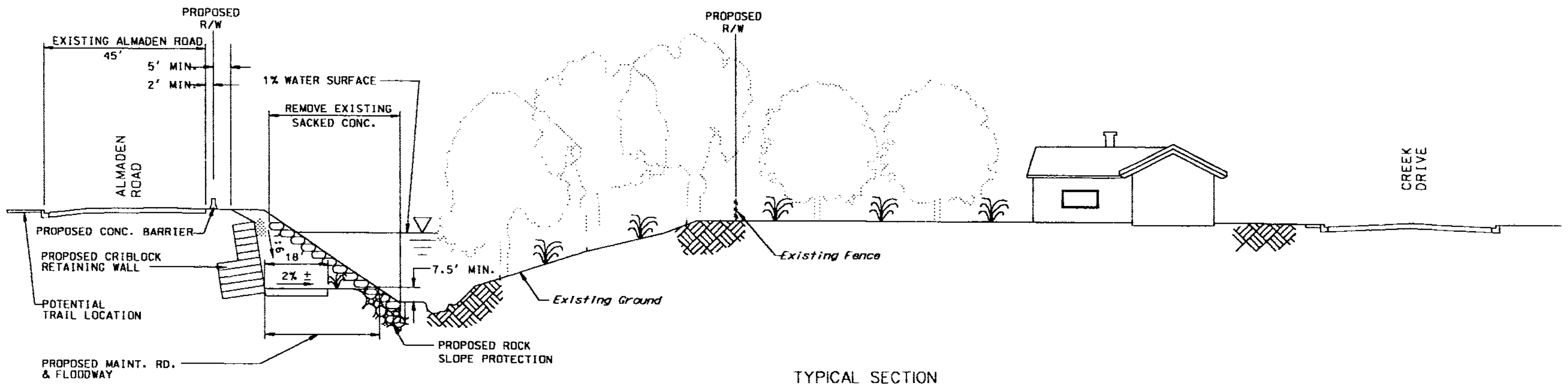
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 785+25± TO STA 806+00±

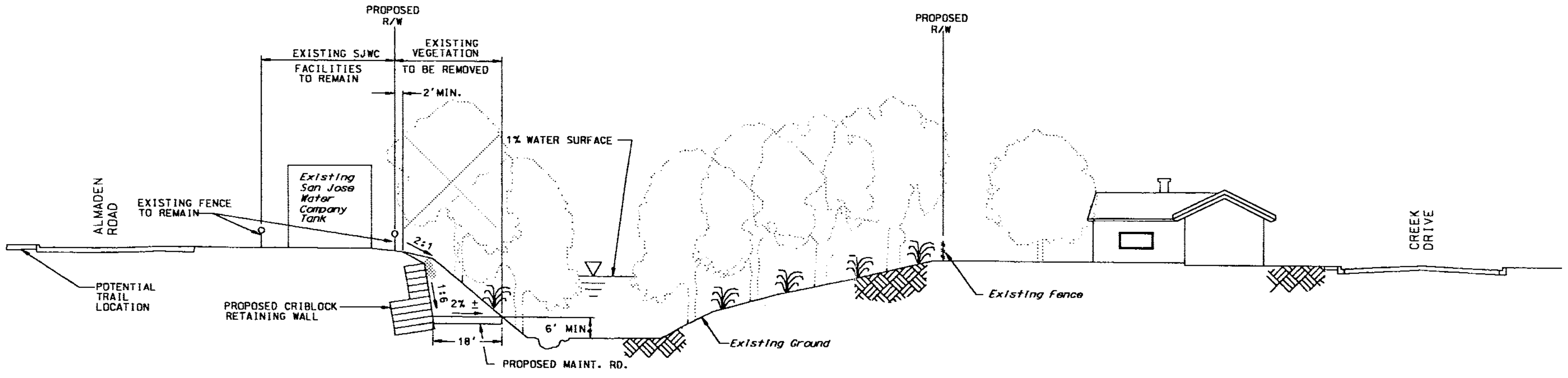
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

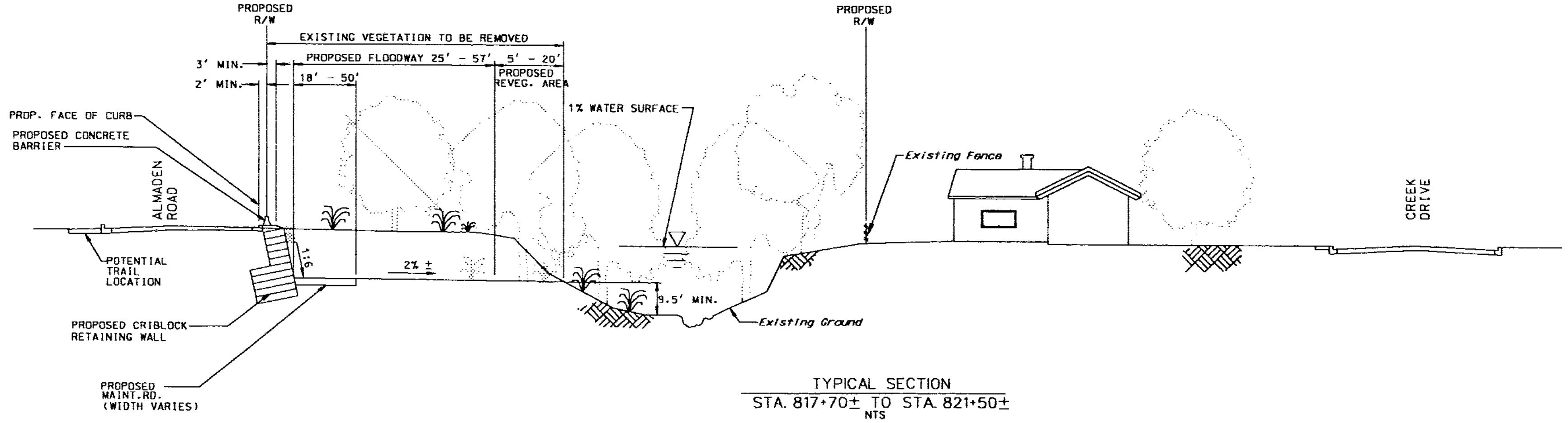
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-18



TYPICAL SECTION
STA. 810+00± TO STA. 812+00±
NTS



TYPICAL SECTION
STA. 815+50± TO STA. 817+00±
NTS



TYPICAL SECTION
STA. 817+70± TO STA. 821+50±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

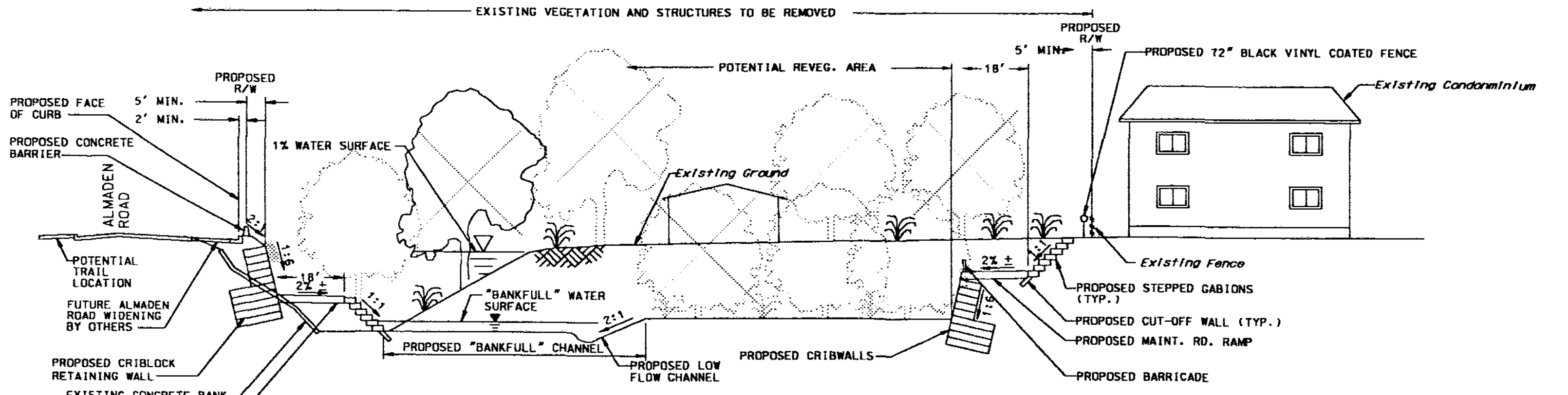
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 810+00± TO STA 821+50±

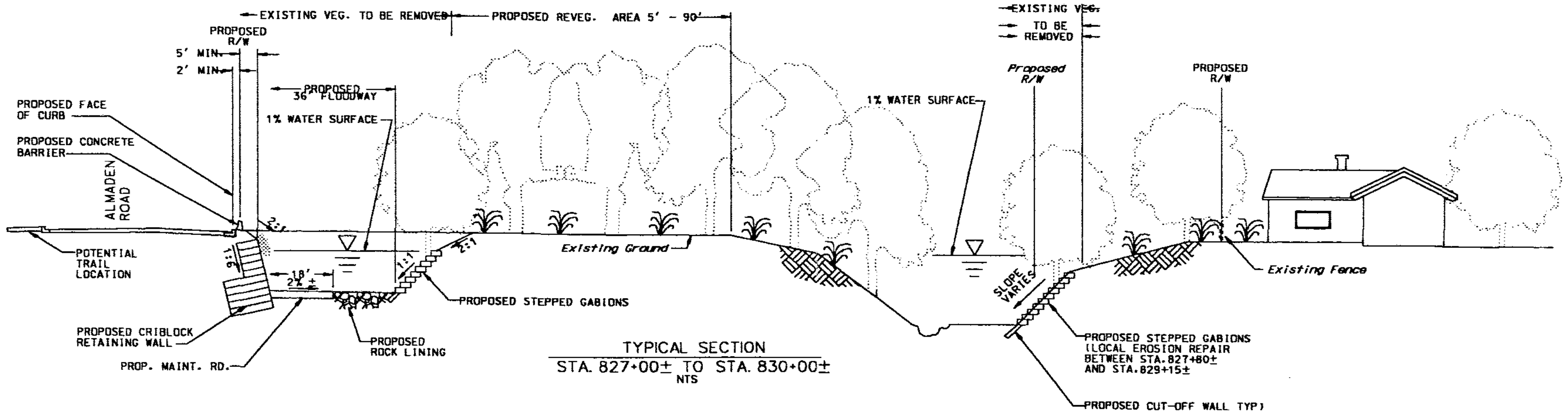
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

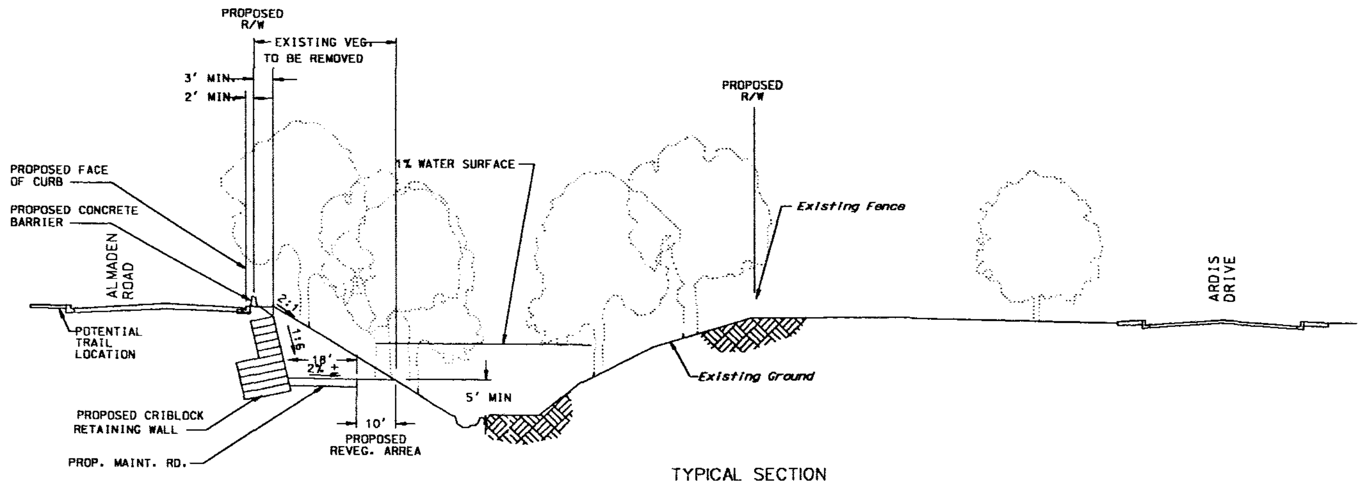
SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-19


EXISTING VEGETATION AND STRUCTURES TO BE REMOVED
PROPOSED R/W
PROPOSED FACE OF CURB
5' MIN.
2' MIN.
PROPOSED CONCRETE BARRIER
1% WATER SURFACE
ALMADEN ROAD
POTENTIAL TRAIL LOCATION
FUTURE ALMADEN ROAD WIDENING BY OTHERS
PROPOSED CRIBLOCK RETAINING WALL
EXISTING CONCRETE BANK TO BE REMOVED
PROP. MAINT. RD.
1.5:1
18'
2% ±
2:1
1:1
"BANKFULL" WATER SURFACE
2:1
PROPOSED "BANKFULL" CHANNEL
PROPOSED LOW FLOW CHANNEL
POTENTIAL REVEG. AREA
Existing Ground
18'
5' MIN
PROPOSED R/W
PROPOSED 72" BLACK VINYL COATED FENCE
Existing Condominium
2% ±
1:1
1.5:1
PROPOSED CRIBWALLS
Existing Fence
PROPOSED STEPPED GABIONS (TYP.)
PROPOSED CUT-OFF WALL (TYP.)
PROPOSED MAINT. RD. RAMP
PROPOSED BARRICADE

TYPICAL SECTION
STA. 822+00± TO STA. 826+50±
NTS



TYPICAL SECTION
STA. 827+00± TO STA. 830+00±
NTS



TYPICAL SECTION
STA. 832+00± TO STA. 842+00±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

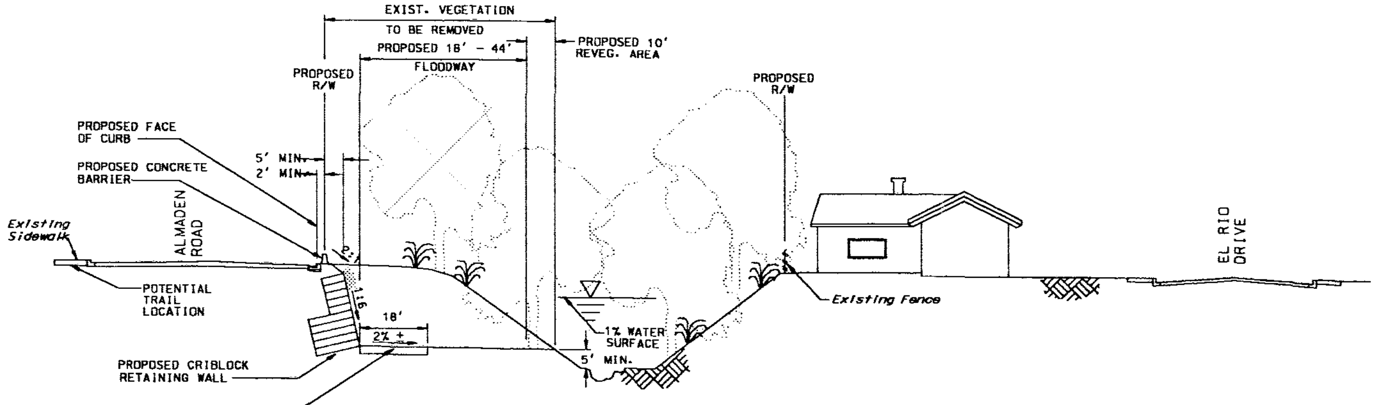
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 822+00± TO STA 844+50±

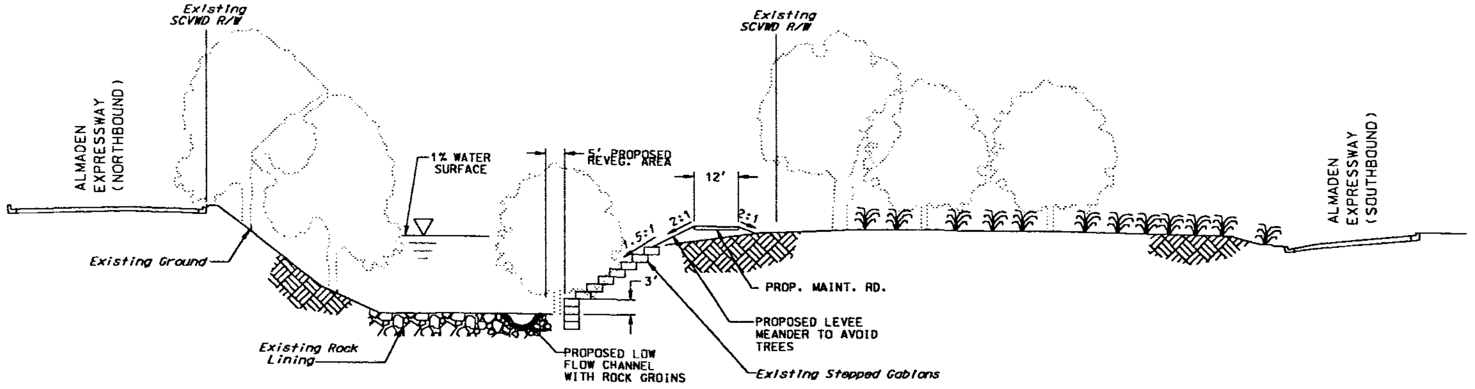
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

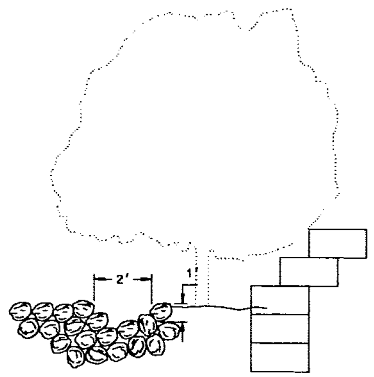
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-20



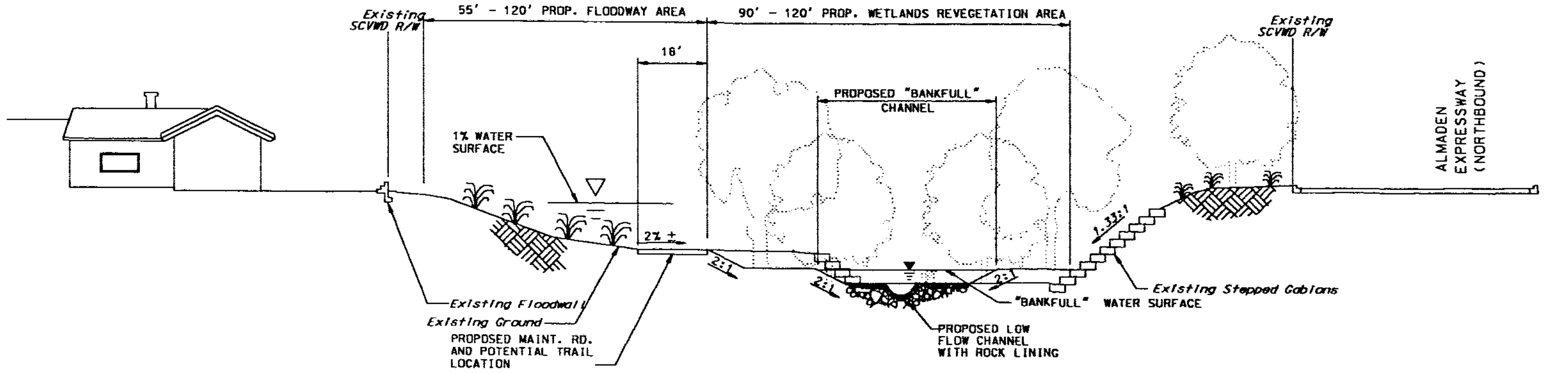
TYPICAL SECTION
STA. 843+50± TO STA. 854+50±
NTS



TYPICAL SECTION
STA. 858+20± TO STA. 868+90±
NTS



LOW FLOW CHANNEL DETAIL
NTS



TYPICAL SECTION
STA. 873+20± TO STA. 885+50±

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

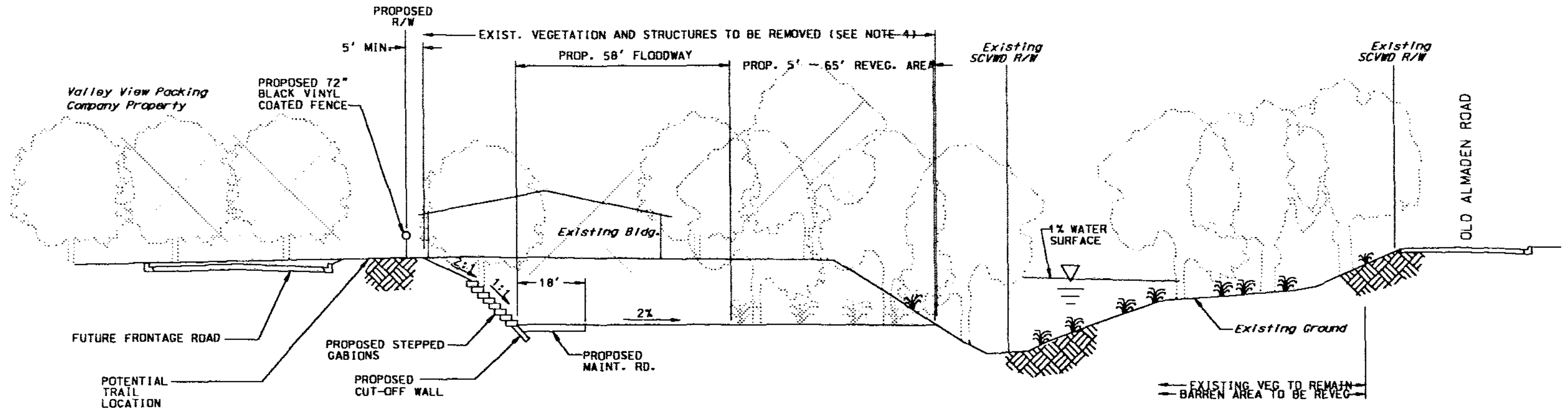
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 858+20± TO STA 885+50±

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
DATE:
CHIEF, PLANNING/ENGINEERING DIVISION

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

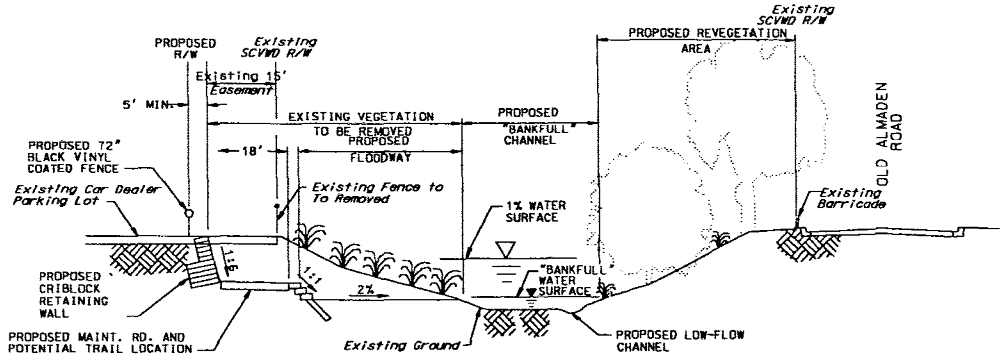
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-21



TYPICAL SECTION

STA. 887+80± TO STA. 903+00±

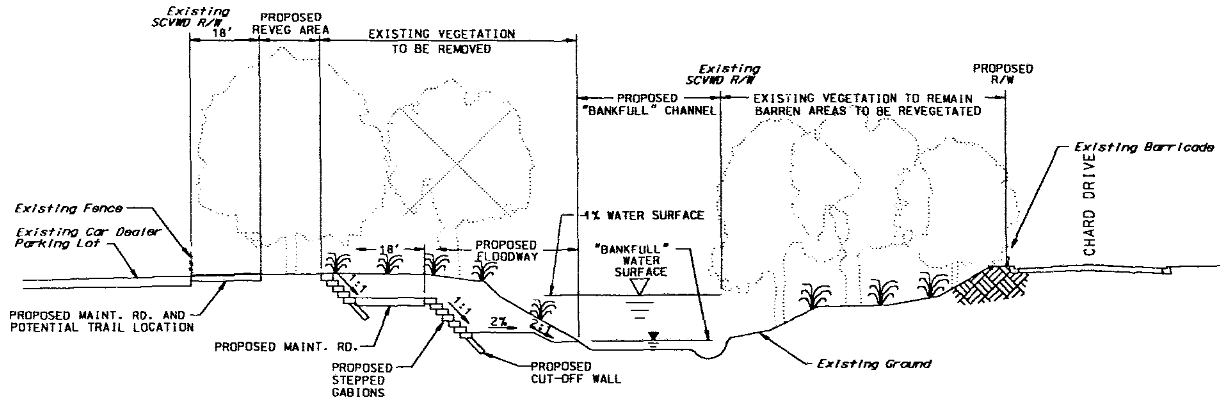
NTS


PROPOSED R/W
Existing SCVWD R/W
Existing 15' Easement
5' MIN.
EXISTING VEGETATION TO BE REMOVED
PROPOSED "BANKFULL" CHANNEL
Existing SCVWD R/W
PROPOSED REVEGETATION AREA
OLD ALMADEN ROAD
18'
PROPOSED FLOODWAY
PROPOSED 72" BLACK VINYL COATED FENCE
Existing Car Dealer Parking Lot
Existing Fence to To Removed
1% WATER SURFACE
Existing Barricade
1:6
1:1
2%
"BANKFULL" WATER SURFACE
PROPOSED CRIBLOCK RETAINING WALL
PROPOSED MAINT. RD. AND POTENTIAL TRAIL LOCATION
Existing Ground
PROPOSED LOW-FLOW CHANNEL

TYPICAL SECTION

STA. 906+15± TO STA. 910+00±

NTS



TYPICAL SECTION

STA. 912+00± TO STA. 915+00±

NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

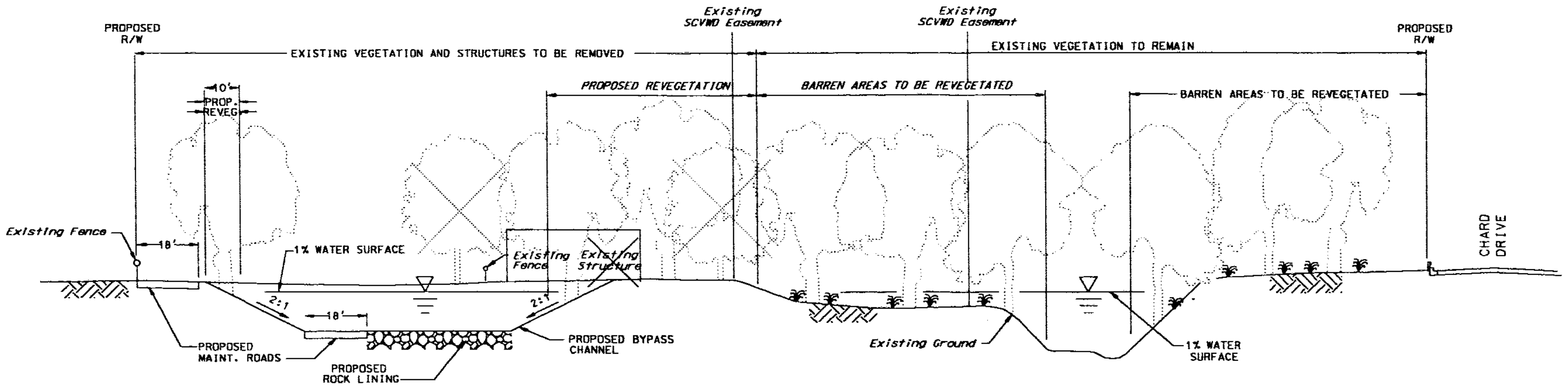
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 888+00± TO STA 917+50±

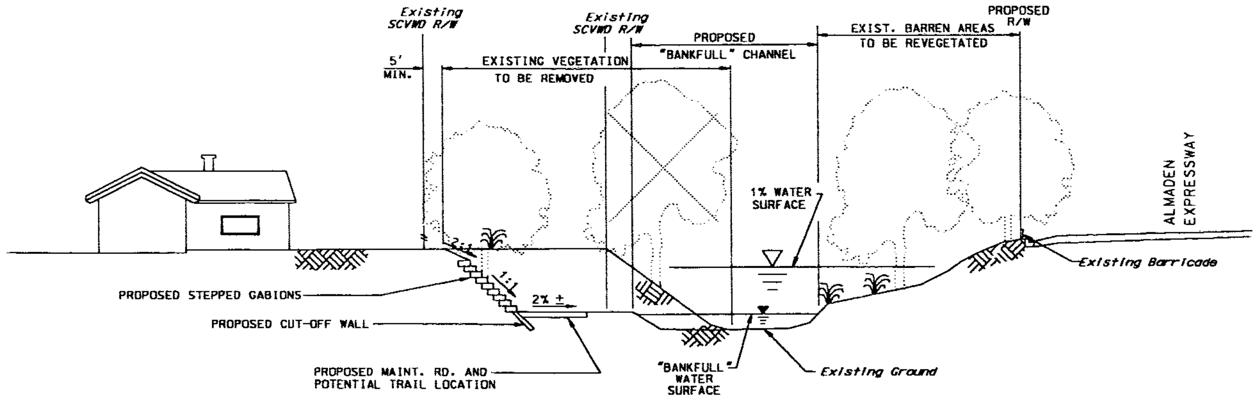
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

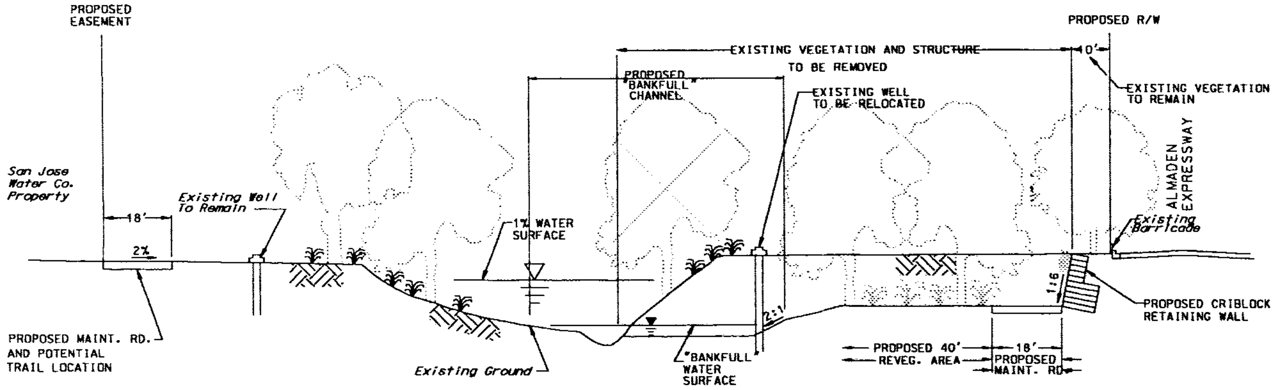
SCALE: 1" = 40'
JOB NO.:
DRAWING NUMBER
SHEET: C-22



TYPICAL SECTION
STA. 917+00± TO STA. 920+00±
NTS



TYPICAL SECTION
STA. 925+00± TO STA. 937+60±
NTS


PROPOSED EASEMENT
EXISTING VEGETATION AND STRUCTURE TO BE REMOVED
PROPOSED R/W
10'
PROPOSED "BANKFULL" CHANNEL
EXISTING WELL TO BE RELOCATED
EXISTING VEGETATION TO REMAIN
ALMADEN EXPRESSWAY
San Jose Water Co. Property
Existing Well To Remain
18'
2%
1% WATER SURFACE
Existing Barricade
2:1
1:6
PROPOSED CRIBLOCK RETAINING WALL
PROPOSED MAINT. RD. AND POTENTIAL TRAIL LOCATION
Existing Ground
"BANKFULL" WATER SURFACE
PROPOSED 40' REVEG. AREA
18'
PROPOSED MAINT. RD.

TYPICAL SECTION
STA. 942+00± TO STA. 948+00±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

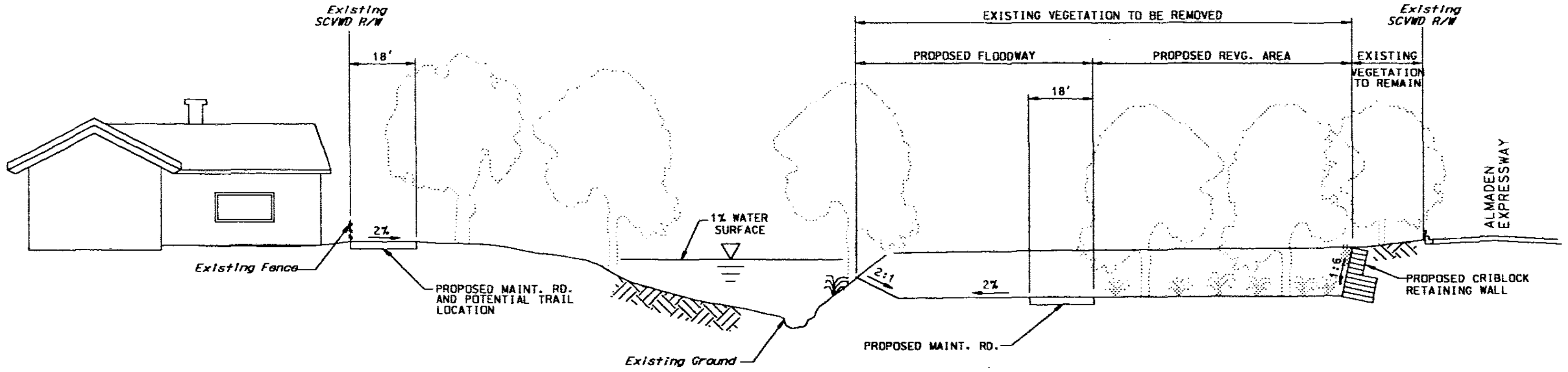
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 917+00± TO STA 948+00±

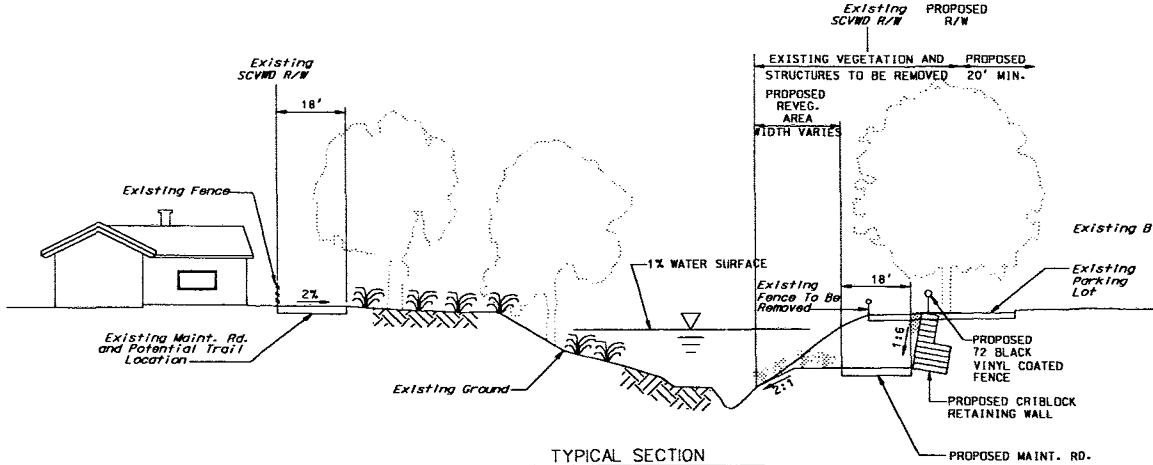
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

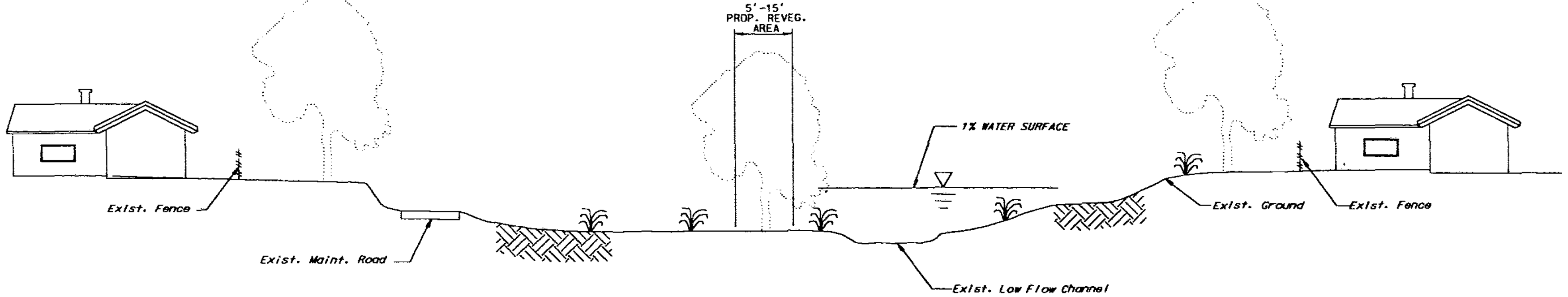
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-23



TYPICAL SECTION
STA. 949+60± TO STA. 951+10±
NTS


Existing SCVWD R/W
18'
Existing Fence
2%
Existing Maint. Rd. and Potential Trail Location
1% WATER SURFACE
Existing Ground
Existing SCVWD R/W
PROPOSED R/W
EXISTING VEGETATION AND STRUCTURES TO BE REMOVED
PROPOSED 20' MIN.
PROPOSED REVEG. AREA WIDTH VARIES
Existing Fence To Be Removed
18'
2:1
1:6
Existing Bldg.
Existing Parking Lot
PROPOSED 72 BLACK VINYL COATED FENCE
PROPOSED CRIBLOCK RETAINING WALL
PROPOSED MAINT. RD.

TYPICAL SECTION
STA. 952+80± TO STA. 954+00±
NTS



TYPICAL SECTION
STA. 964+00 TO STA. 968+20±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

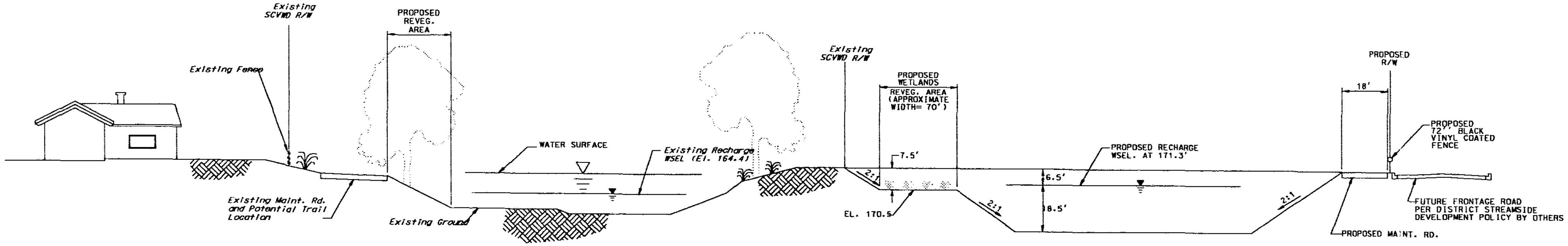
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 949+60 TO STA 968+20

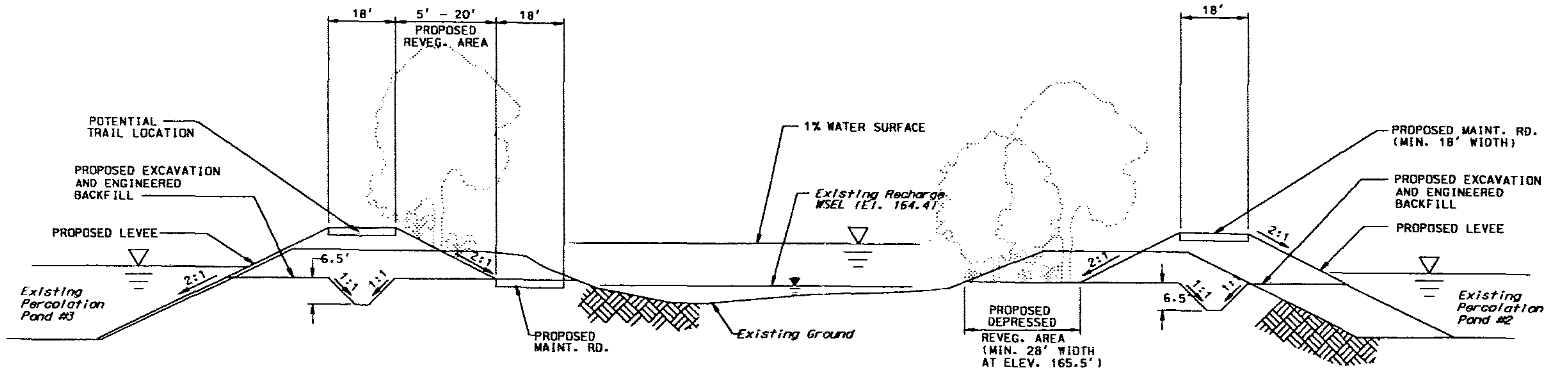
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

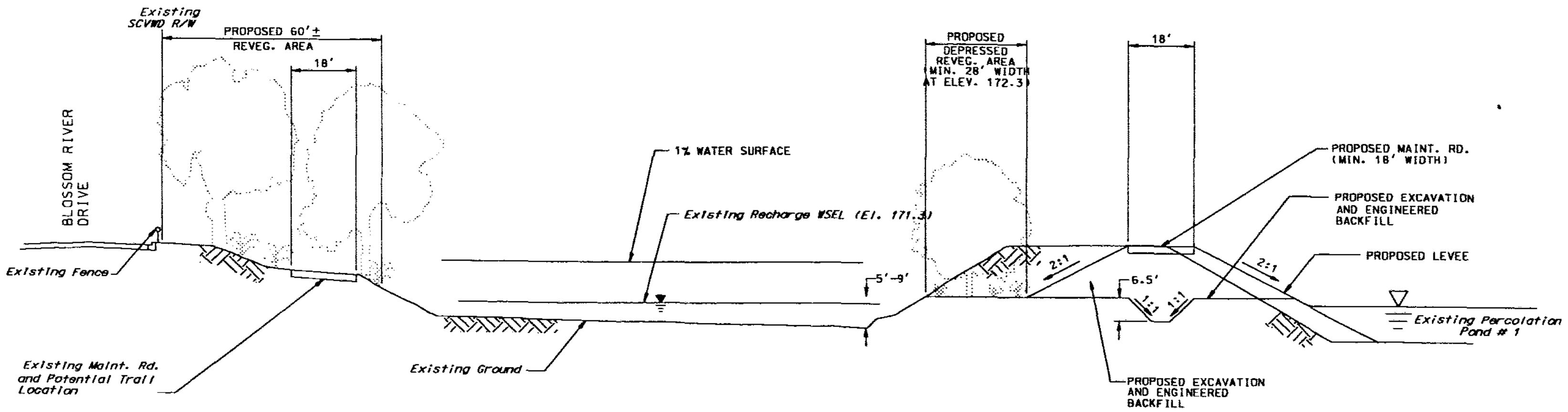
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-24


Existing SCVWD R/W
PROPOSED REVEG. AREA
Existing Fence
WATER SURFACE
Existing Recharge WSEL (El. 164.4)
Existing Maint. Rd. and Potential Trail Location
Existing Ground
Existing SCVWD R/W
PROPOSED WETLANDS
REVEG. AREA (APPROXIMATE WIDTH= 70')
7.5'
2:1
EL. 170.5
PROPOSED RECHARGE WSEL. AT 171.3'
6.5'
8.5'
2:1
2:1
PROPOSED R/W
18'
PROPOSED 72'' BLACK VINYL COATED FENCE
FUTURE FRONTAGE ROAD PER DISTRICT STREAMSIDE DEVELOPMENT POLICY BY OTHERS
PROPOSED MAINT. RD.

TYPICAL SECTION
STA. 976+50± TO STA. 985+00±
NTS


18'
5' - 20'
18'
PROPOSED REVEG. AREA
POTENTIAL TRAIL LOCATION
PROPOSED EXCAVATION AND ENGINEERED BACKFILL
PROPOSED LEVEE
Existing Percolation Pond #3
2:1
6.5'
1:1
1:1
2:1
PROPOSED MAINT. RD.
1% WATER SURFACE
Existing Recharge WSEL (El. 164.4)
Existing Ground
PROPOSED DEPRESSED REVEG. AREA (MIN. 28' WIDTH AT ELEV. 165.5')
18'
PROPOSED MAINT. RD. (MIN. 18' WIDTH)
PROPOSED EXCAVATION AND ENGINEERED BACKFILL
PROPOSED LEVEE
2:1
2:1
6.5
1:1
1:1
Existing Percolation Pond #2

TYPICAL SECTION
STA. 989+00± TO STA. 998+00±
NTS


Existing SCVWD R/W
PROPOSED 60'± REVEG. AREA
18'
BLOSSOM RIVER DRIVE
Existing Fence
Existing Maint. Rd. and Potential Trail Location
1% WATER SURFACE
Existing Recharge WSEL (El. 171.3)
5'-9'
Existing Ground
PROPOSED DEPRESSED REVEG. AREA (MIN. 28' WIDTH AT ELEV. 172.3)
18'
2:1
6.5'
1:1
1:1
2:1
PROPOSED MAINT. RD. (MIN. 18' WIDTH)
PROPOSED EXCAVATION AND ENGINEERED BACKFILL
PROPOSED LEVEE
Existing Percolation Pond # 1
PROPOSED EXCAVATION AND ENGINEERED BACKFILL

TYPICAL SECTION
STA. 999+00± TO STA. 1012+00±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

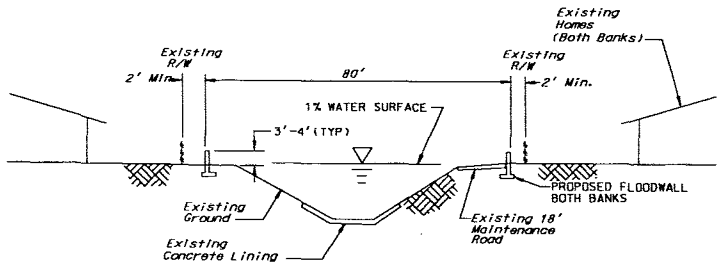
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
TYPICAL SECTIONS
STA 952+80± TO STA 1012+00±

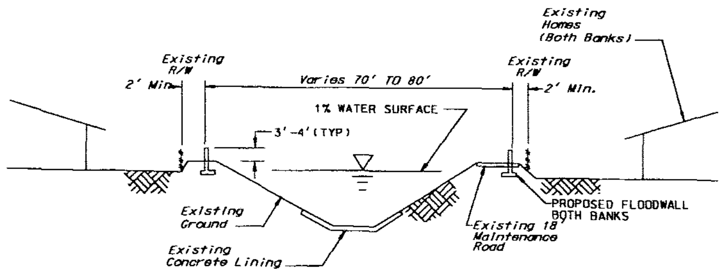
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

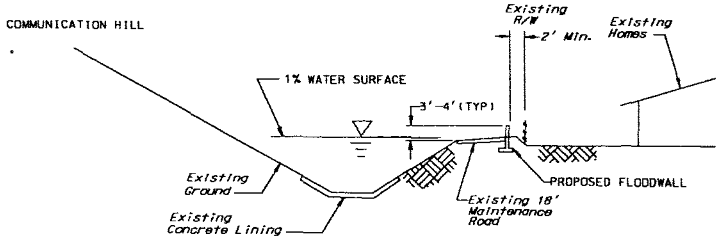
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-25



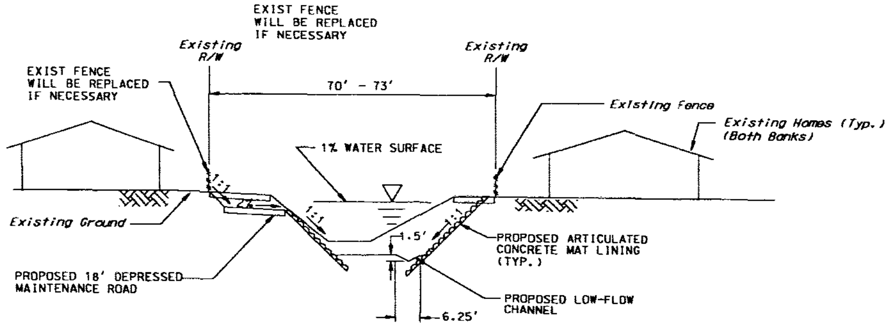
TYPICAL SECTION (CANOAS CREEK)
STA.2+00± TO STA.5+50±
NTS



TYPICAL SECTION (CANOAS CREEK)
STA. 5+50± TO STA. 13+40±
NTS



TYPICAL SECTION (CANOAS CREEK)
STA. 15+60± TO STA. 28+85±
NTS



TYPICAL SECTION (ROSS CREEK)
STA. 4+70± TO STA. 50+20±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

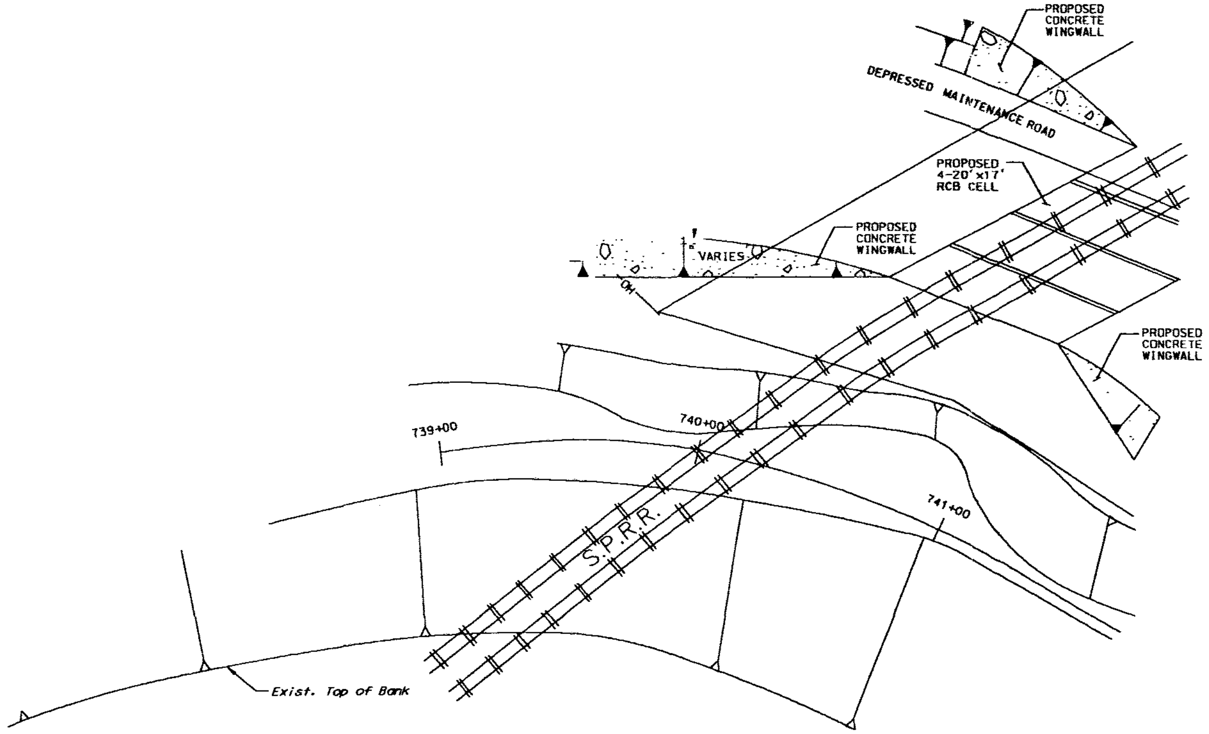
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
CANOAS CREEK & ROSS CREEK
TYPICAL SECTIONS

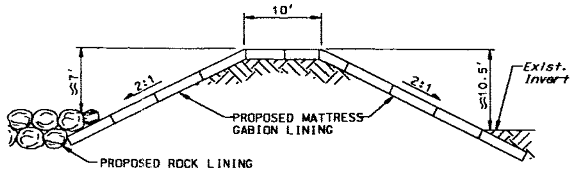
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

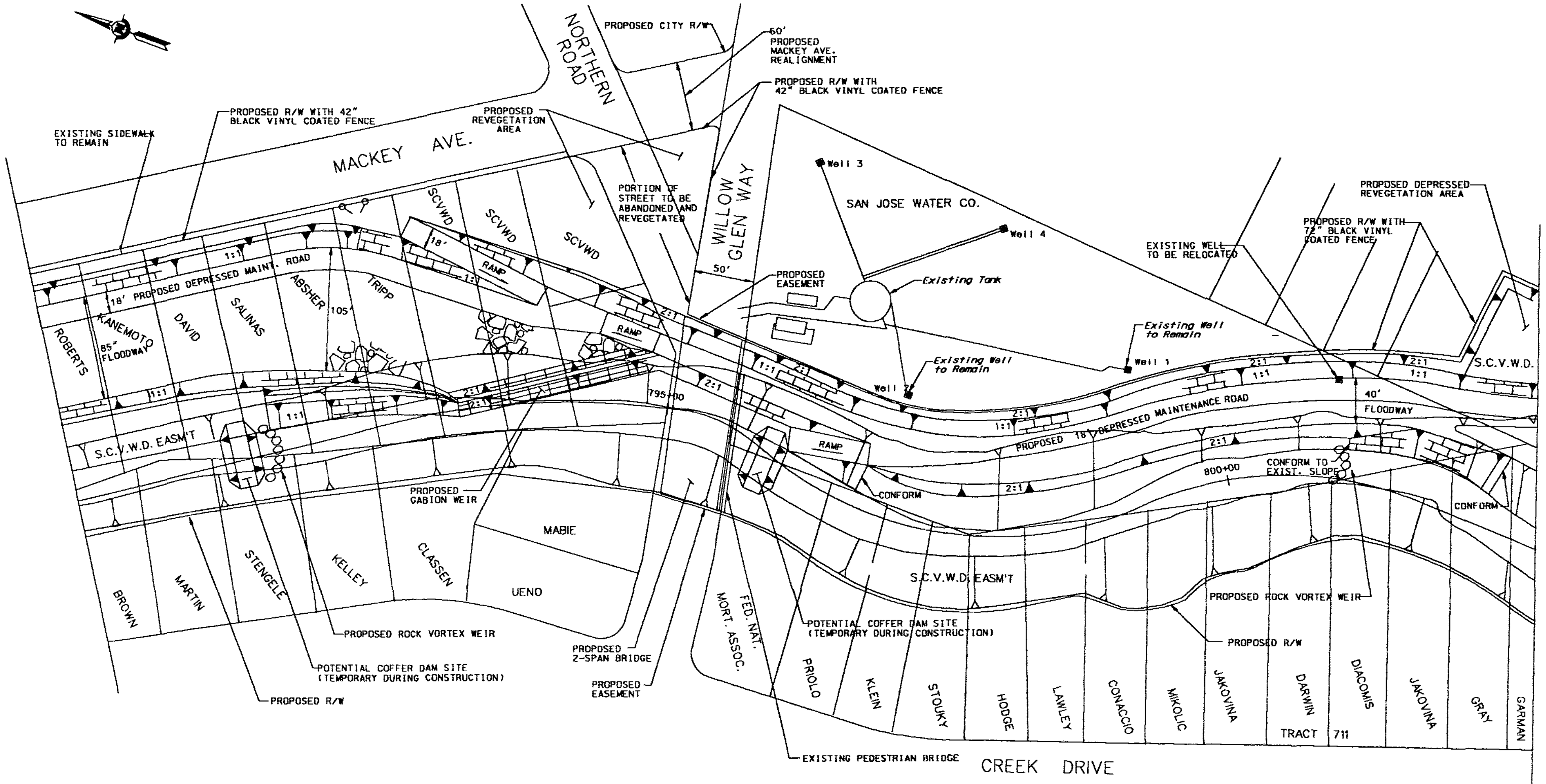
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-26


PROPOSED CONCRETE WINGWALL
DEPRESSED MAINTENANCE ROAD
PROPOSED 4-20'x17' RCB CELL
VARIES
PROPOSED CONCRETE WINGWALL
PROPOSED CONCRETE WINGWALL
739+00
740+00
741+00
S.P.R.R.
Exist. Top of Bank

PLAN AT BEGINNING OF PROJECT
SCALE: 1"=30'


10'
2:1
2:1
≤7'
≤10.5'
Exist. Invert
PROPOSED MATTRESS GABION LINING
PROPOSED ROCK LINING FOR GABION CHANNEL

SECTION OF GABION WEIR
SCALE: 1"=100'



BYPASS CHANNEL DIVERSION PLAN
SCALE: 1"=50'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

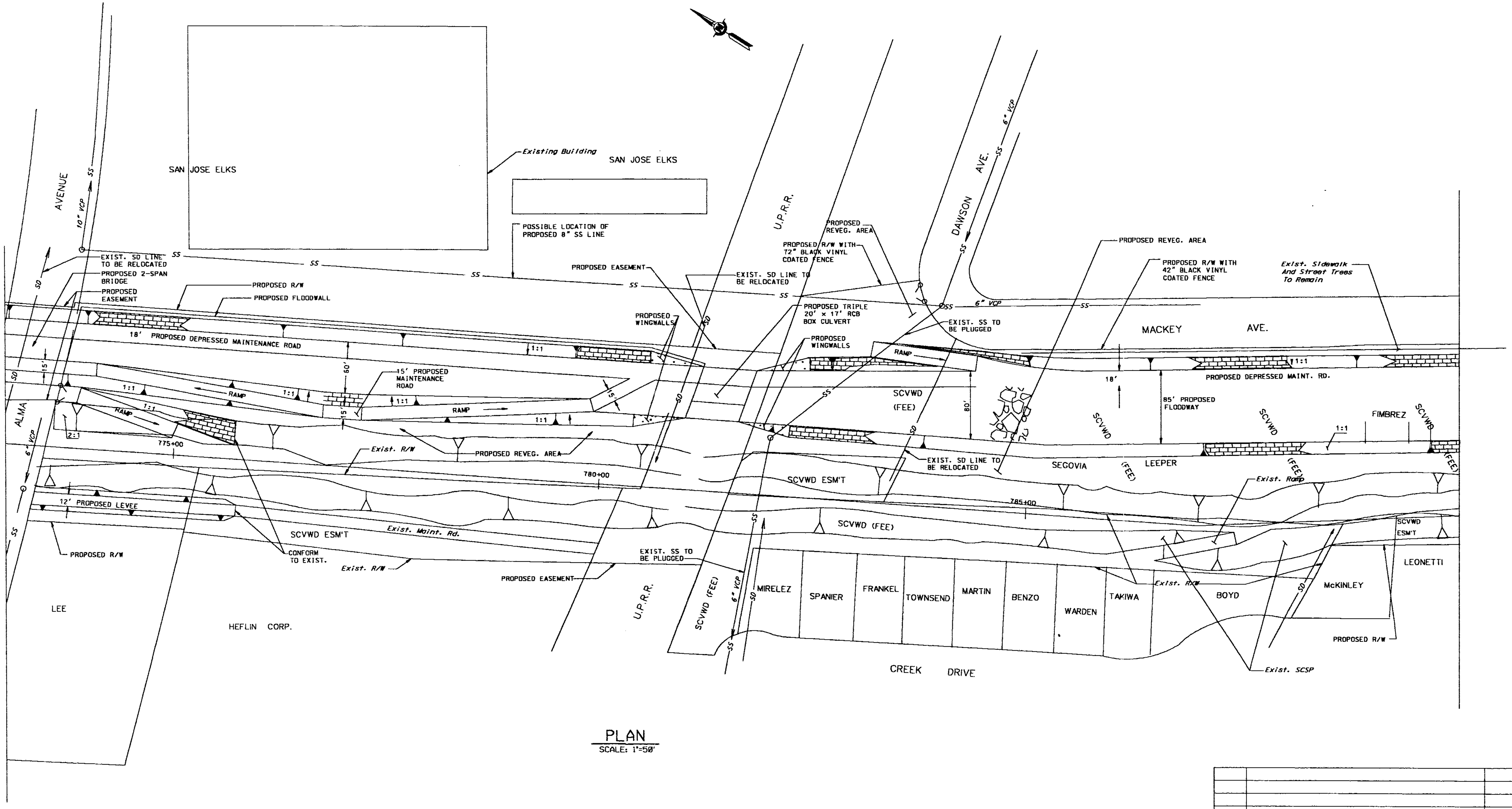
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN DETAIL
BEGINNING OF PROJECT & BYPASS CHANNEL

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-27



PLAN
SCALE: 1"=50'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

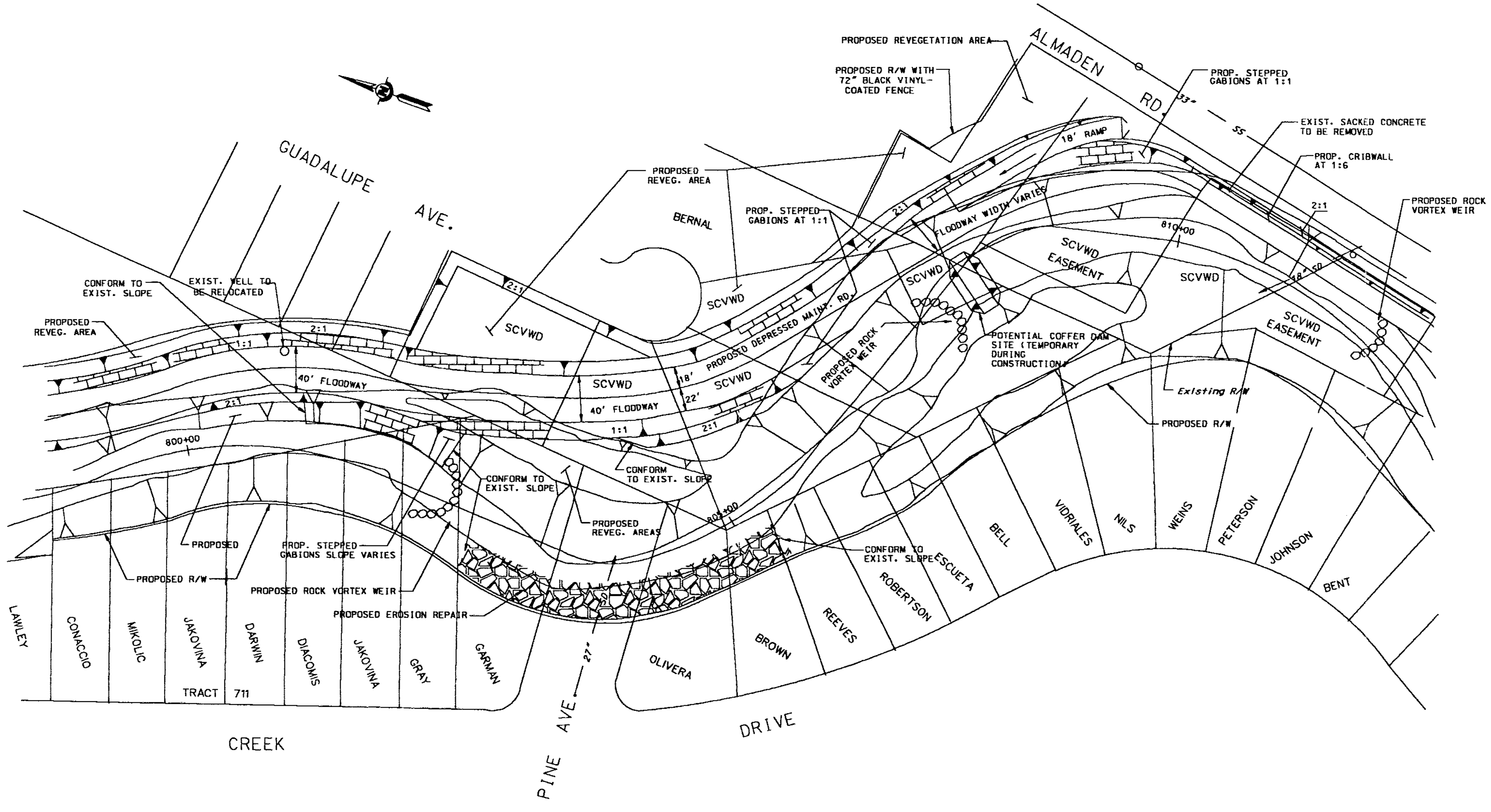
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN DETAIL
ELKS LODGE OPEN BYPASS

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-28



PLAN

SCALE: 1"=50'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

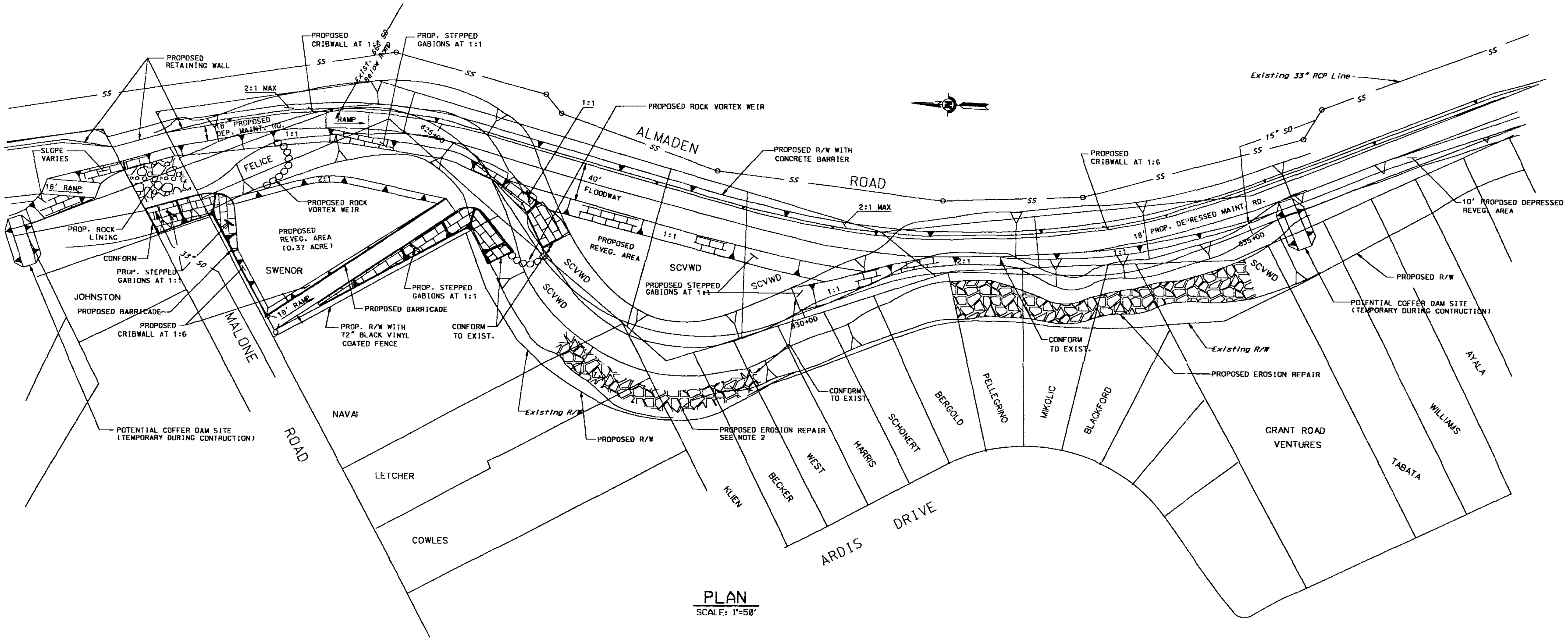
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN DETAIL
PINE AVENUE ISLAND

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-29



PLAN
SCALE: 1"=50'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

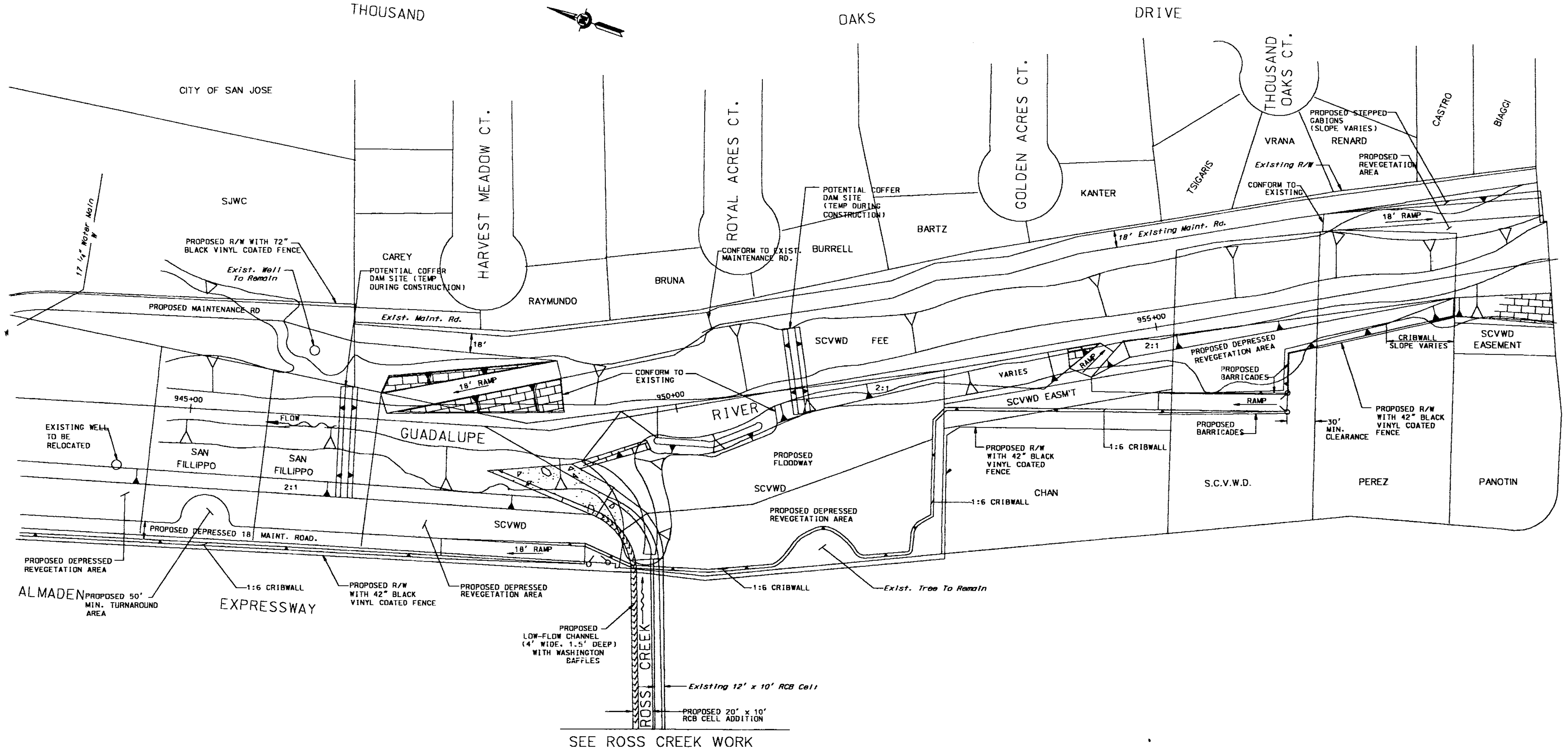
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN DETAIL
MALONE ROAD ISLAND

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: 1" = 40'
JOB NO.:
DRAWING NUMBER
SHEET: C-30


THOUSAND
OAKS
DRIVE
CITY OF SAN JOSE
SJWC
17 1/4" Water Main
HARVEST MEADOW CT.
ROYAL ACRES CT.
GOLDEN ACRES CT.
THOUSAND OAKS CT.
CAREY
RAYMUNDO
BRUNA
BURRELL
BARTZ
KANTER
TSIGARIS
VRANA
RENARD
CASTRO
BIAGGI
PROPOSED R/W WITH 72" BLACK VINYL COATED FENCE
Exist. Well To Remain
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
CONFORM TO EXIST. MAINTENANCE RD.
POTENTIAL COFFER DAM SITE (TEMP DURING CONSTRUCTION)
PROPOSED STEPPED GABIONS (SLOPE VARIES)
Existing R/W
PROPOSED REVEGETATION AREA
CONFORM TO EXISTING
18' RAMP
18' Existing Maint. Rd.
PROPOSED MAINTENANCE RD
Exist. Maint. Rd.
18'
955+00
SCVWD FEE
2:1
PROPOSED DEPRESSED REVEGETATION AREA
CRIBWALL SLOPE VARIES
SCVWD EASEMENT
18' RAMP
CONFORM TO EXISTING
950+00
VARIES
RAMP
PROPOSED BARRICADES
945+00
FLOW
RIVER
2:1
SCVWD EASM'T
RAMP
PROPOSED R/W WITH 42" BLACK VINYL COATED FENCE
30' MIN. CLEARANCE
PROPOSED BARRICADES
EXISTING WELL TO BE RELOCATED
SAN FILLIPPO
SAN FILLIPPO
GUADALUPE
PROPOSED FLOODWAY
PROPOSED R/W WITH 42" BLACK VINYL COATED FENCE
1:6 CRIBWALL
CHAN
S.C.V.W.D.
PEREZ
PANOTIN
2:1
SCVWD
1:6 CRIBWALL
SCVWD
PROPOSED DEPRESSED REVEGETATION AREA
PROPOSED DEPRESSED 18' MAINT. ROAD.
18' RAMP
PROPOSED DEPRESSED REVEGETATION AREA
ALMADEN
PROPOSED 50' MIN. TURNAROUND AREA
1:6 CRIBWALL
EXPRESSWAY
PROPOSED R/W WITH 42" BLACK VINYL COATED FENCE
PROPOSED DEPRESSED REVEGETATION AREA
1:6 CRIBWALL
Exist. Tree To Remain
PROPOSED LOW-FLOW CHANNEL (4' WIDE, 1.5' DEEP) WITH WASHINGTON BAFFLES
ROSS CREEK
Existing 12' x 10' RCB Cell
PROPOSED 20' x 10' RCB CELL ADDITION
SEE ROSS CREEK WORK

PLAN

SCALE: 1"=50'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

DRAWN BY:
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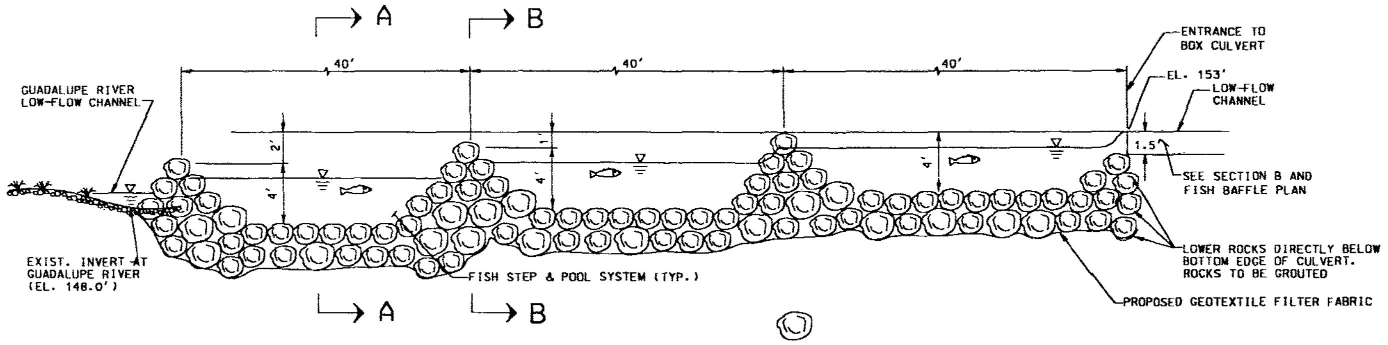
SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLAN DETAIL
ROSS CREEK CONFLUENCE

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

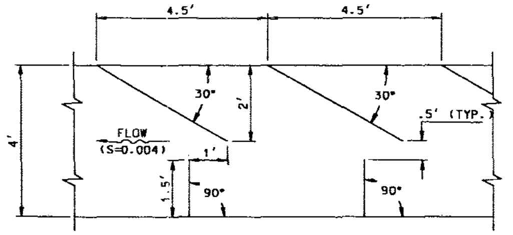
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-31


A
B
40'
40'
40'
GUADALUPE RIVER LOW-FLOW CHANNEL
ENTRANCE TO BOX CULVERT
EL. 153'
LOW-FLOW CHANNEL
1.5'
SEE SECTION B AND FISH BAFFLE PLAN
2'
4'
1'
4'
4'
EXIST. INVERT AT GUADALUPE RIVER (EL. 148.0')
FISH STEP & POOL SYSTEM (TYP.)
LOWER ROCKS DIRECTLY BELOW BOTTOM EDGE OF CULVERT. ROCKS TO BE GROUTED
PROPOSED GEOTEXTILE FILTER FABRIC
A
B

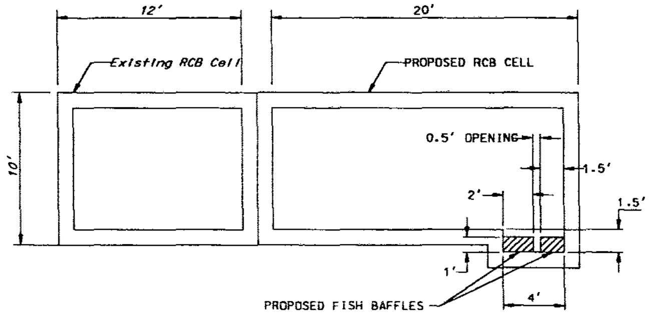
LONGITUDINAL SECTION

SCALE: 1"=5'



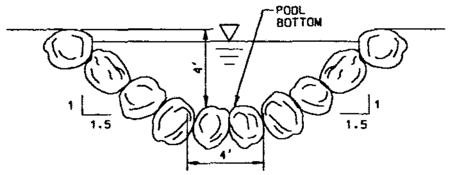
DETAIL- FISH BAFFLE PLAN

SCALE: 1"=2'



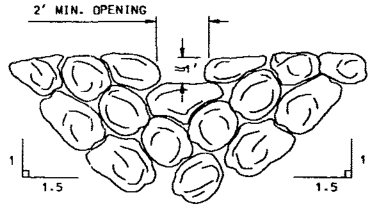
BOX CULVERT SECTION

SCALE: 1"=5



SECTION A-A

SCALE: 1"=4'



SECTION B-B

SCALE: 1"=5'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

DRAWN BY:
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DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

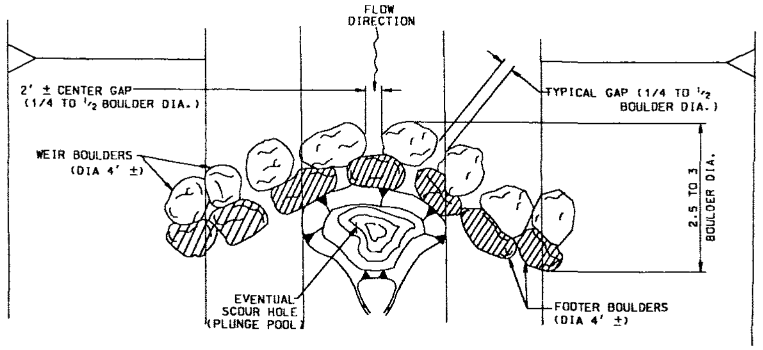
SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
SECTIONS AND DETAIL
ROSS CREEK CONFLUENCE

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH

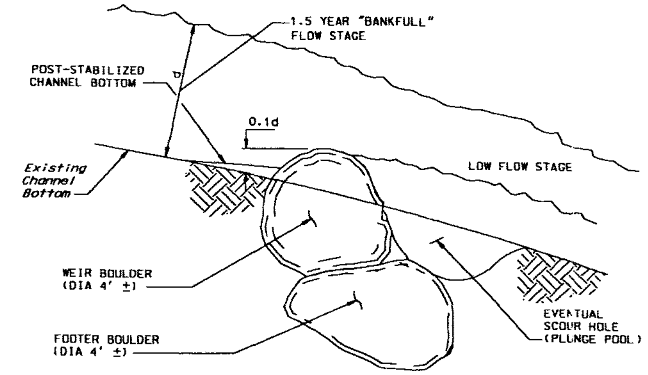
APPROVED: DATE:
CHIEF, PLANNING/ENGINEERING DIVISION

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

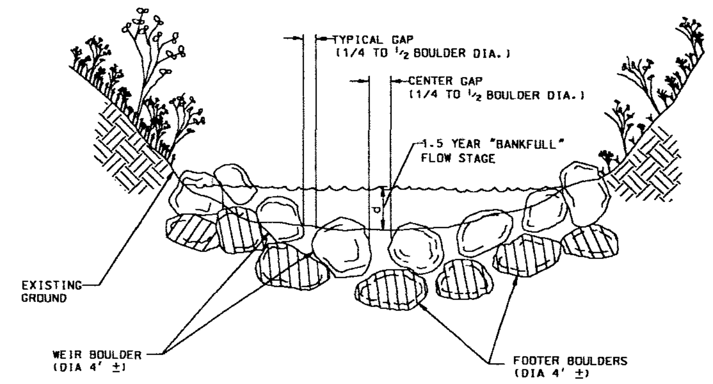
SCALE: 1" = 40'
JOB NO.:
DRAWING NUMBER
SHEET: C-32


FLOW DIRECTION
2' ± CENTER GAP (1/4 TO 1/2 BOULDER DIA.)
TYPICAL GAP (1/4 TO 1/2 BOULDER DIA.)
WEIR BOULDERS (DIA 4' ±)
2.5 TO 3 BOULDER DIA.
EVENTUAL SCOUR HOLE (PLUNGE POOL)
FOOTER BOULDERS (DIA 4' ±)

PLAN
NTS


1.5 YEAR "BANKFULL" FLOW STAGE
POST-STABILIZED CHANNEL BOTTOM
d
0.1d
Existing Channel Bottom
LOW FLOW STAGE
WEIR BOULDER (DIA 4' ±)
FOOTER BOULDER (DIA 4' ±)
EVENTUAL SCOUR HOLE (PLUNGE POOL)

PROFILE
NTS


TYPICAL GAP (1/4 TO 1/2 BOULDER DIA.)
CENTER GAP (1/4 TO 1/2 BOULDER DIA.)
1.5 YEAR "BANKFULL" FLOW STAGE
d
EXISTING GROUND
WEIR BOULDER (DIA 4' ±)
FOOTER BOULDERS (DIA 4' ±)

TYPICAL SECTION
(LOOKING UPSTREAM)
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

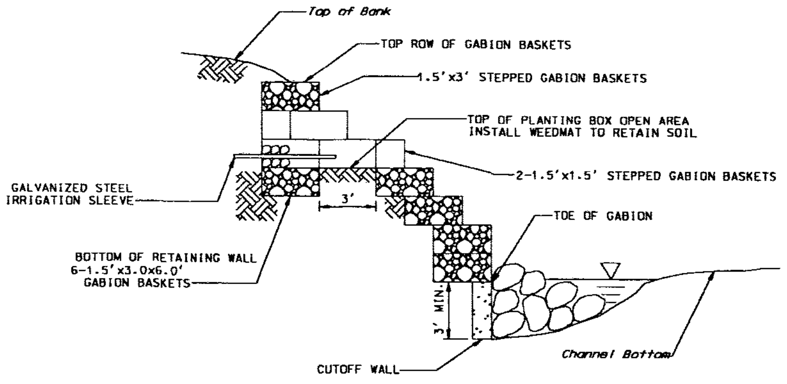
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
VORTEX ROCK WEIR DETAIL
PLAN, PROFILE & SECTION

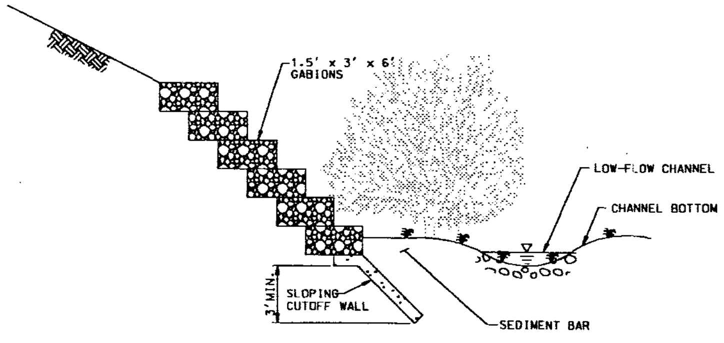
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

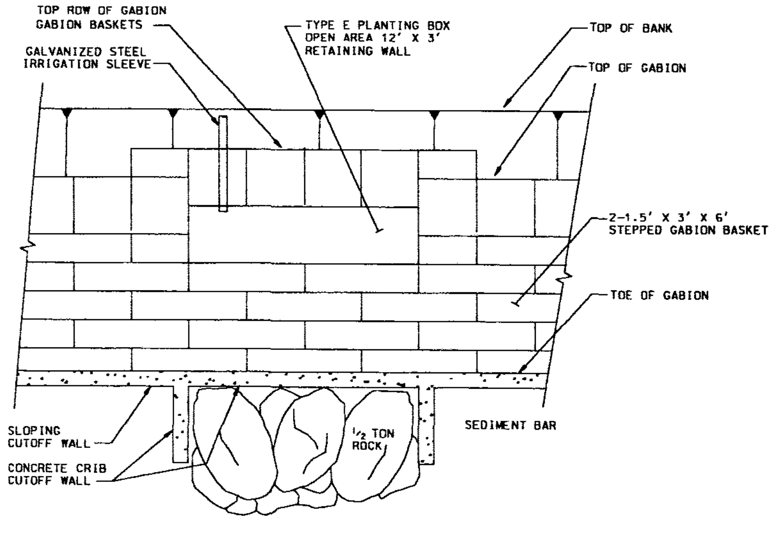
SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-33


Top of Bank
TOP ROW OF GABION BASKETS
1.5'x3' STEPPED GABION BASKETS
TOP OF PLANTING BOX OPEN AREA
INSTALL WEEDMAT TO RETAIN SOIL
2-1.5'x1.5' STEPPED GABION BASKETS
GALVANIZED STEEL
IRRIGATION SLEEVE
3'
TOE OF GABION
BOTTOM OF RETAINING WALL
6-1.5'x3.0x6.0'
GABION BASKETS
3' MIN.
Channel Bottom
CUTOFF WALL

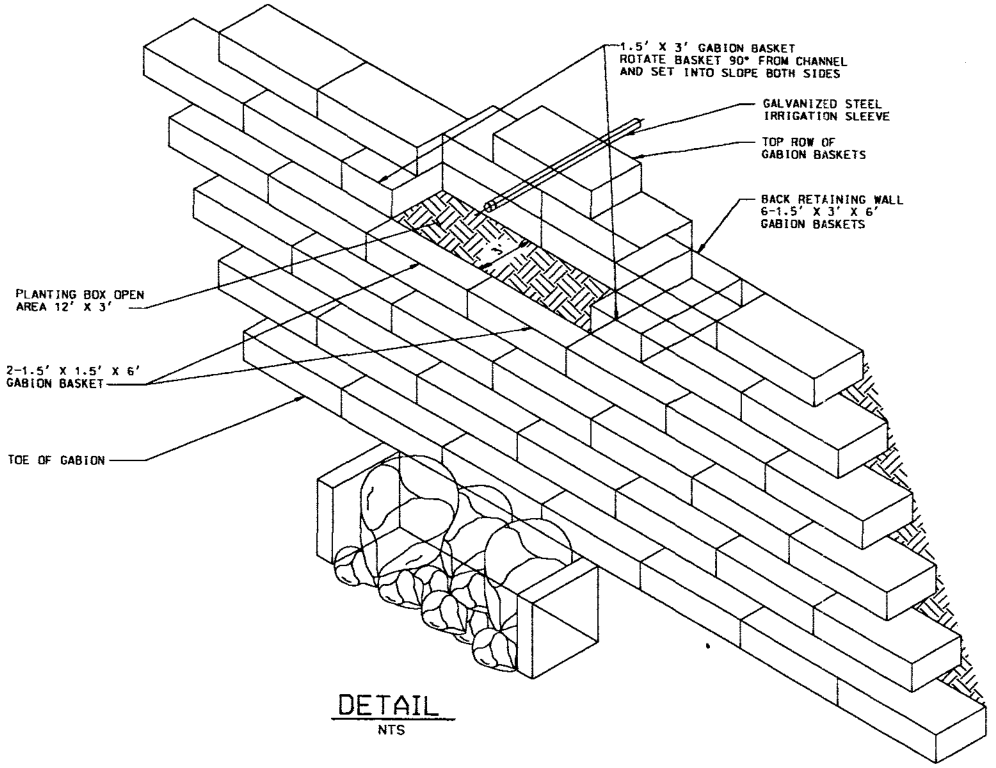
SECTION
SCALE: 1"=100'



SECTION
SCALE: 1"=4'



PLAN
SCALE: 1"=100'



DETAIL
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

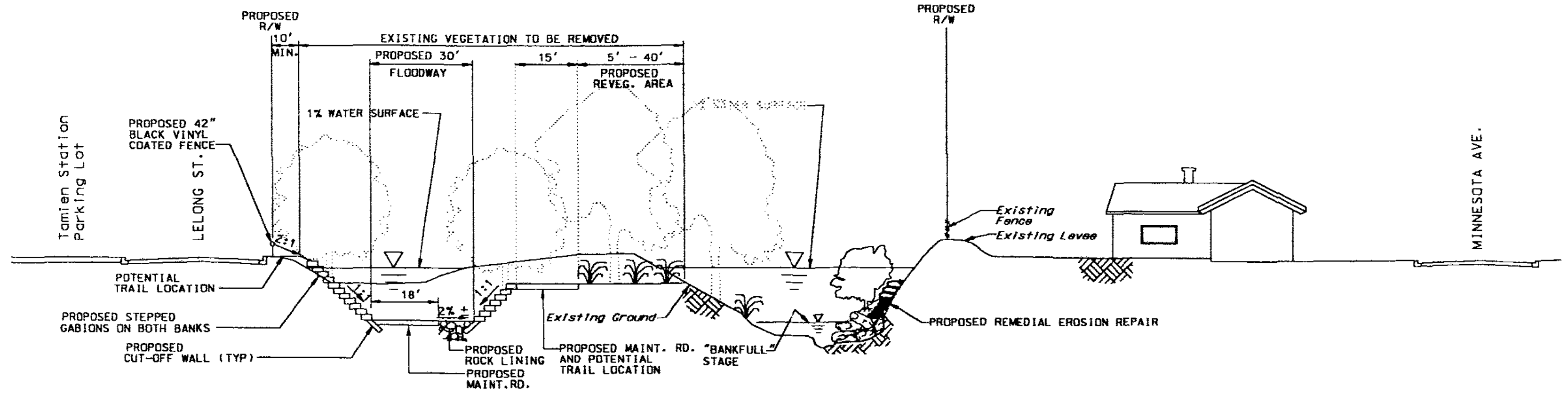
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
PLANTING BOX
PLAN, SECTIONS & DETAIL

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

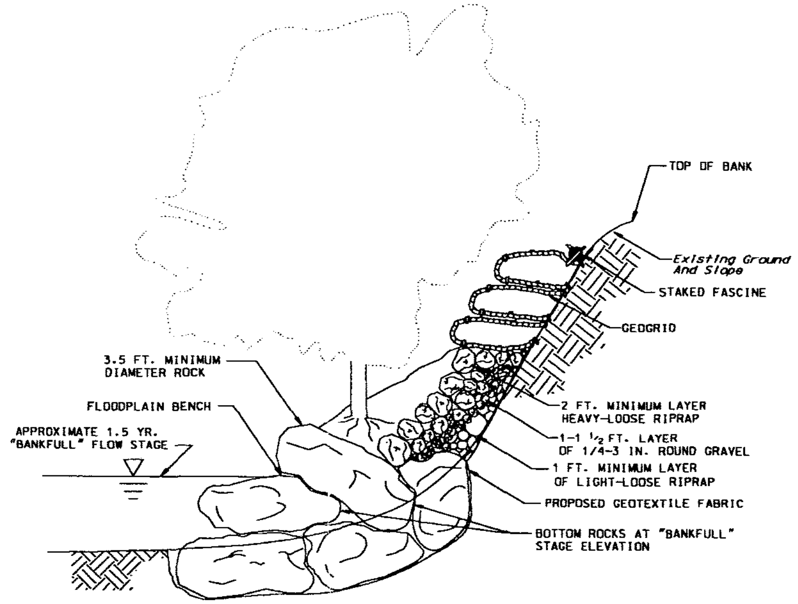
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-34



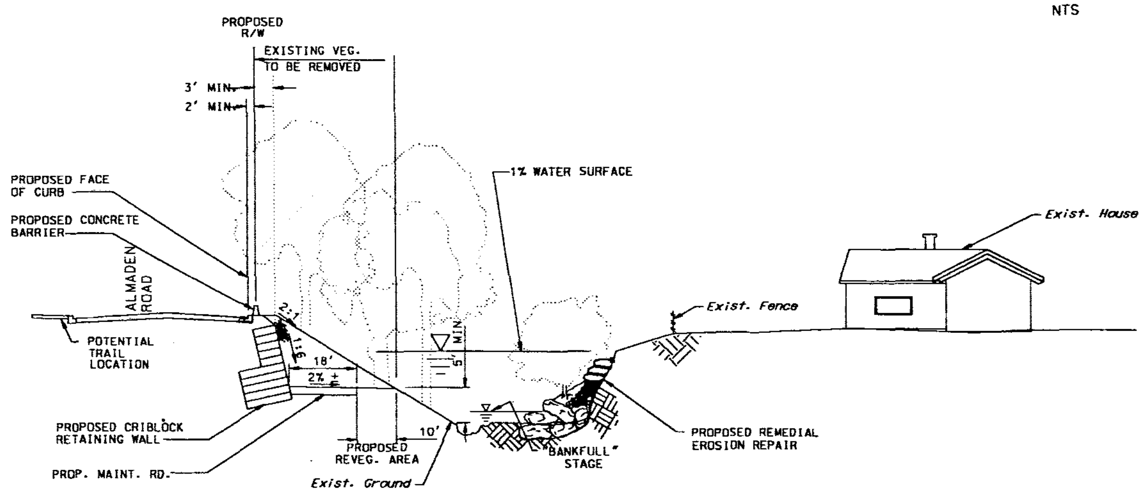
STA 759+50± TO STA 764+50±
POTENTIAL EROSION REPAIR METHOD

(LOOKING UPSTREAM)
NTS


TOP OF BANK
Existing Ground And Slope
STAKED FASCINE
GEOGRID
3.5 FT. MINIMUM DIAMETER ROCK
FLOODPLAIN BENCH
APPROXIMATE 1.5 YR. "BANKFULL" FLOW STAGE
2 FT. MINIMUM LAYER HEAVY-LOOSE RIPRAP
1-1 ½ FT. LAYER OF 1/4-3 IN. ROUND GRAVEL
1 FT. MINIMUM LAYER OF LIGHT-LOOSE RIPRAP
PROPOSED GEOTEXTILE FABRIC
BOTTOM ROCKS AT "BANKFULL" STAGE ELEVATION

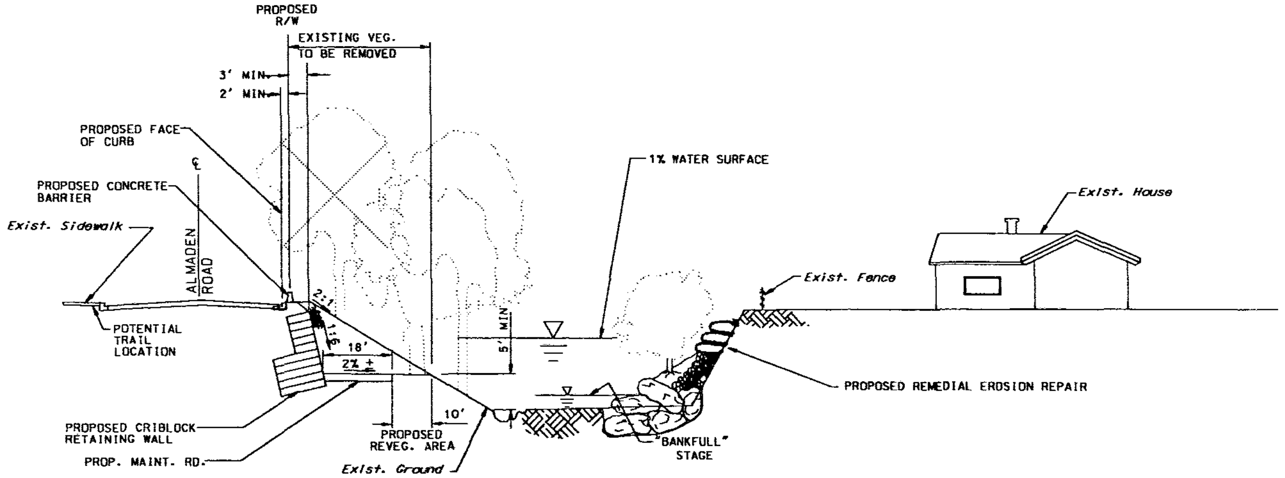
INTEGRATED BIOTECHNICAL EROSION REPAIR DETAIL

NTS



STA 831+70 ± TO STA 834+70 ±
POTENTIAL EROSION REPAIR METHOD

(LOOKING UPSTREAM)
NTS



STA 839+80± TO STA 842+00±
POTENTIAL EROSION REPAIR METHOD

(LOOKING UPSTREAM)
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

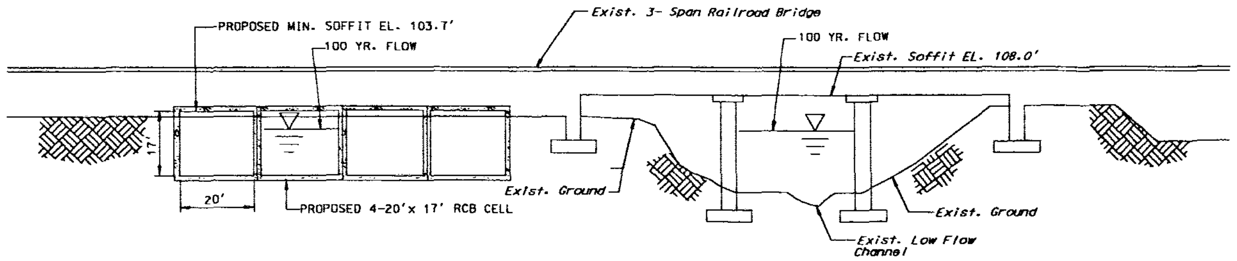
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
SITES NEEDING EROSION REPAIR

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

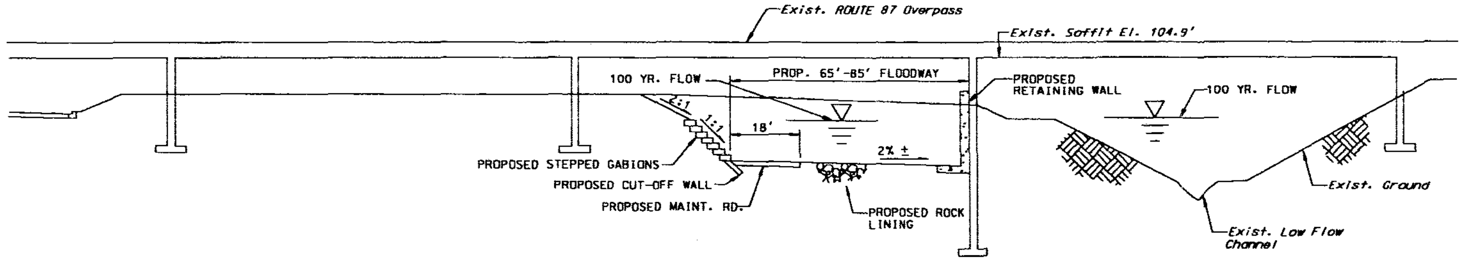
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: NTS
JOB NO.:
DRAWING NUMBER
SHEET: C-35



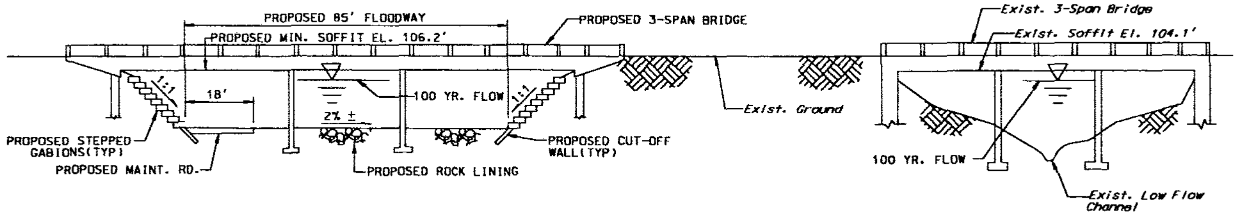
SPRR SECTION

STA. 738+80± TO STA. 741+30±
NTS



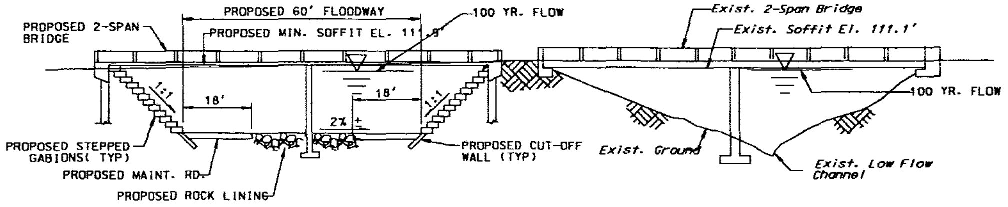
ROUTE 87 OVERCROSSING

STA. 742+25± TO STA. 744+15±
NTS



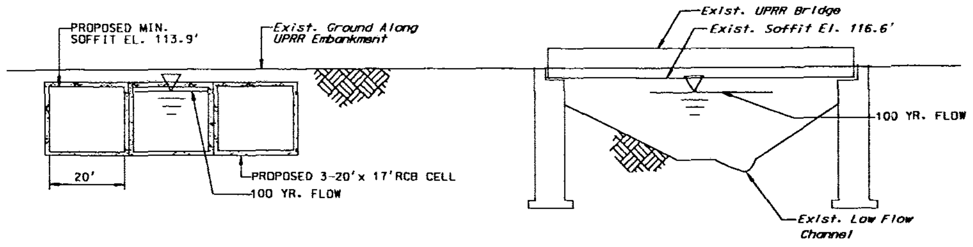
WILLOW STREET BRIDGES

STA. 747+00± TO STA. 750+20
NTS



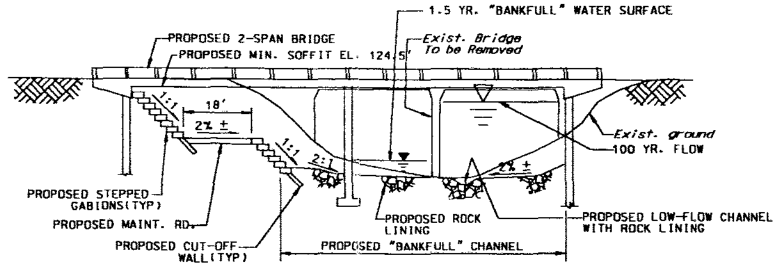
ALMA STREET BRIDGES

STA. 772+70± TO STA. 773+60±
NTS


PROPOSED MIN. SOFFIT EL. 113.9'
Exist. Ground Along UPRR Embankment
20'
PROPOSED 3-20'x 17'RCB CELL
100 YR. FLOW
Exist. UPRR Bridge
Exist. Soffit El. 116.6'
100 YR. FLOW
Exist. Low Flow Channel

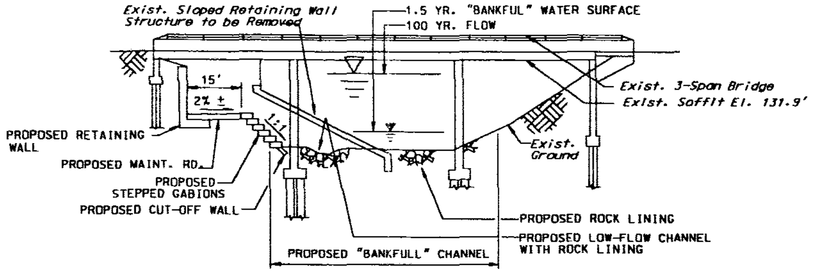
UPRR BRIDGE

STA. 780+70± TO STA. 781+60±
NTS


1.5 YR. "BANKFULL" WATER SURFACE
PROPOSED 2-SPAN BRIDGE
PROPOSED MIN. SOFFIT EL. 124.5'
Exist. Bridge To be Removed
1:1
18'
2% ±
1:1
2:1
2% ±
Exist. ground
100 YR. FLOW
PROPOSED STEPPED GABIONS(TYP)
PROPOSED MAINT. RD.
PROPOSED CUT-OFF WALL(TYP)
PROPOSED ROCK LINING
PROPOSED LOW-FLOW CHANNEL WITH ROCK LINING
PROPOSED "BANKFULL" CHANNEL

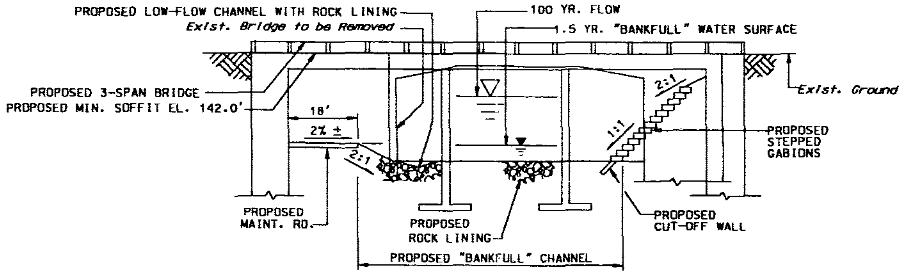
WILLOW GLEN WAY BRIDGE

STA. 792+20± TO STA. 795+50±
NTS



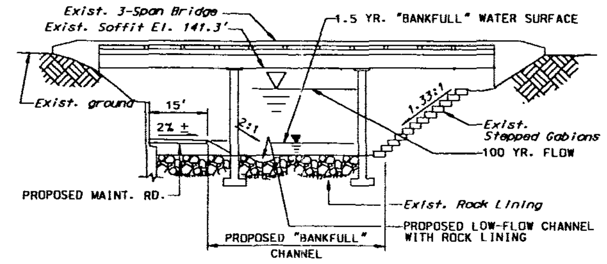
MALONE ROAD BRIDGE

STA.821+50± TO STA. 822+00±
NTS


PROPOSED LOW-FLOW CHANNEL WITH ROCK LINING
Exist. Bridge to be Removed
100 YR. FLOW
1.5 YR. "BANKFULL" WATER SURFACE
PROPOSED 3-SPAN BRIDGE
PROPOSED MIN. SOFFIT EL. 142.0'
18'
2% ±
2:1
2:1
1:1
Exist. Ground
PROPOSED STEPPED GABIONS
PROPOSED MAINT. RD.
PROPOSED ROCK LINING
PROPOSED CUT-OFF WALL
PROPOSED "BANKFULL" CHANNEL

CURTNER AVENUE BRIDGE

STA. 842+50± TO STA. 843+25±
NTS



ALMADEN EXPRESSWAY BRIDGE (SOUTHBOUND)

STA. 855+65± TO STA. 856+50±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

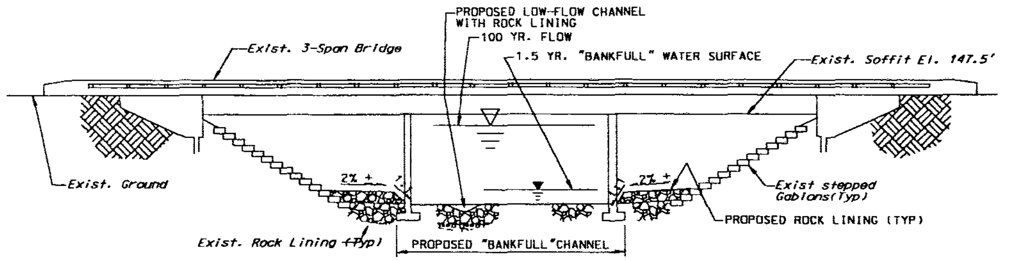
DRAWN BY:
CHECKED BY:
DESIGNED BY:
PROJECT COORDINATOR
SUBMITTED:
CHIEF, CIVIL DESIGN SECTION

SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
BRIDGE AND CULVERTS
STA. 738+80 TO STA. 856+50

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

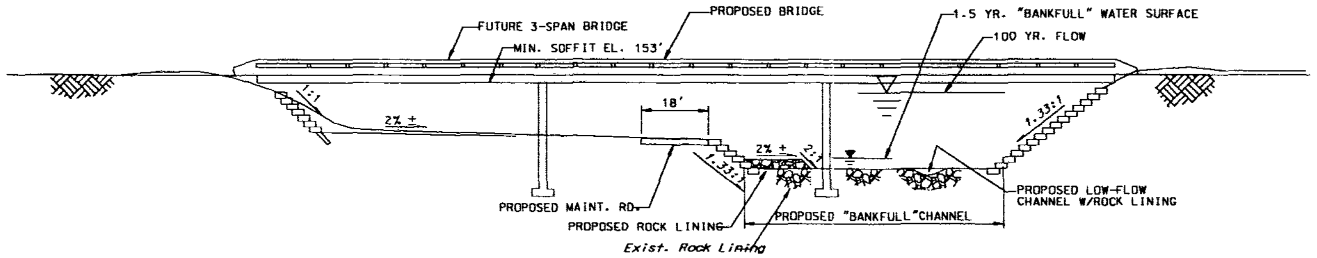
PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: AS SHOWN
JOB NO.:
DRAWING NUMBER
SHEET: C-36


PROPOSED LOW-FLOW CHANNEL WITH ROCK LINING
100 YR. FLOW
1.5 YR. "BANKFULL" WATER SURFACE
Exist. 3-Span Bridge
Exist. Soffit El. 147.5'
Exist. Ground
2% ±
Exist stepped Gablons(Typ)
PROPOSED ROCK LINING (TYP)
Exist. Rock Lining ~~(Typ)~~
PROPOSED "BANKFULL" CHANNEL

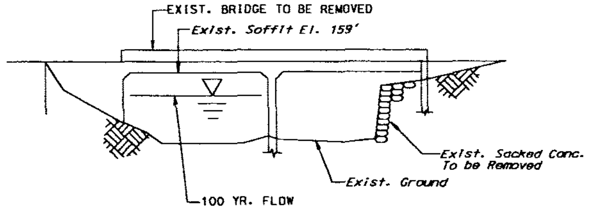
ALMADEN EXPWY. BRIDGE (NORTHBOUND)

STA. 870+00± TO STA. 873+20±
NTS



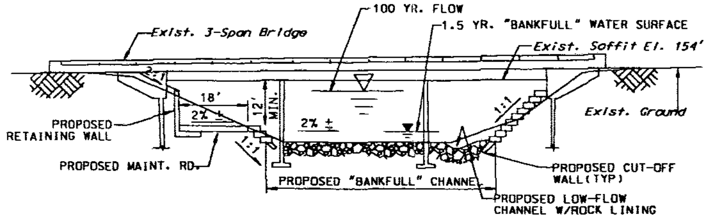
PEARL AVE. BRIDGE

STA. 885+10± TO STA. 885+80±
NTS


EXIST. BRIDGE TO BE REMOVED
Exist. Soffit El. 159'
Exist. Sacked Conc. To be Removed
Exist. Ground
100 YR. FLOW

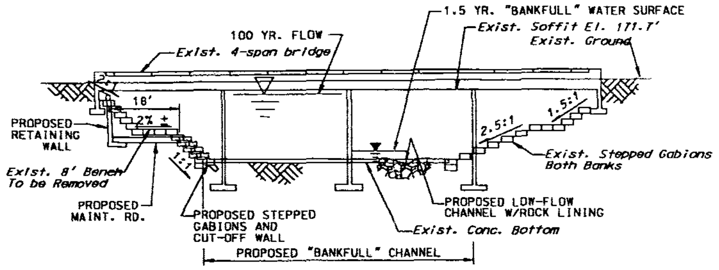
HILLSDALE AVE. BRIDGE

STA. 907+10± TO STA. 907+40±
NTS



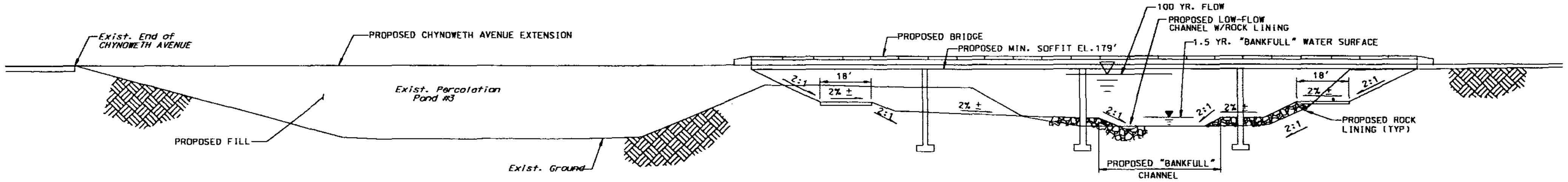
CAPITOL EXPWY. BRIDGE

STA. 909+95± TO STA. 911+35±
NTS


1.5 YR. "BANKFULL" WATER SURFACE
100 YR. FLOW
Exist. 4-span bridge
Exist. Soffit El. 171.7'
Exist. Ground
2:1
18'
2% ±
1:1
1.5:1
2.5:1
PROPOSED RETAINING WALL
Exist. 8' Bench To be Removed
PROPOSED MAINT. RD.
Exist. Stepped Gabions Both Banks
PROPOSED STEPPED GABIONS AND CUT-OFF WALL
PROPOSED LOW-FLOW CHANNEL W/ROCK LINING
Exist. Conc. Bottom
PROPOSED "BANKFULL" CHANNEL

BRANHAM LANE BRIDGE

STA. 958+95± TO STA. 960+30±
NTS


Exist. End of CHYNOWETH AVENUE
PROPOSED CHYNOWETH AVENUE EXTENSION
Exist. Percolation Pond #3
PROPOSED FILL
Exist. Ground
PROPOSED BRIDGE
PROPOSED MIN. SOFFIT EL.179'
100 YR. FLOW
PROPOSED LOW-FLOW CHANNEL W/ROCK LINING
1.5 YR. "BANKFULL" WATER SURFACE
2:1
18'
2% ±
2:1
2% ±
2:1
2:1
2% ±
18'
2% ±
2:1
2:1
PROPOSED ROCK LINING (TYP)
PROPOSED "BANKFULL" CHANNEL

CHYNOWETH AVENUE BRIDGE

STA. 986+60± TO STA. 987+70±
NTS

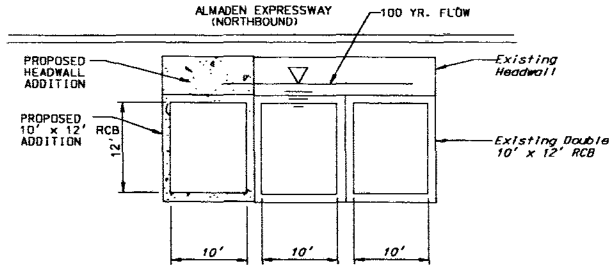
NOTE:
THIS BRIDGE NOT INCLUDED
IN PROPOSED PROJECT

SYMBOL	DESCRIPTION	DATE	APPROVAL

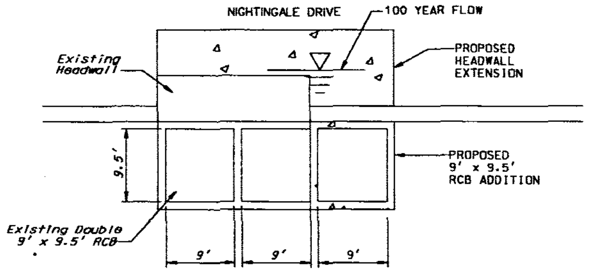
REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

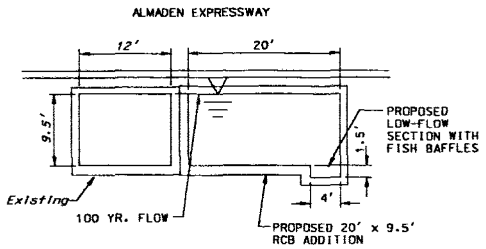
DRAWN BY:	SANTA CLARA COUNTY CALIFORNIA UPPER GUADALUPE RIVER FLOOD CONTROL PROJECT FEASIBILITY DESIGN *BRIDGES AND CULVERTS* ***STA. 870+00 TO STA. 987+70***
CHECKED BY:	
DESIGNED BY: PROJECT COORDINATOR	
SUBMITTED: CHIEF, CIVIL DESIGN SECTION	
APPROVAL RECOMMENDED: CHIEF, ENGINEERING BRANCH	APPROVED: DATE: CHIEF, PLANNING/ENGINEERING DIVISION
PREPARED UNDER THE DIRECTION OF MICHAEL J. WALSH LT. COLONEL C.E., DISTRICT ENGINEER	SCALE: AS SHOWN JOB NO.: DRAWING NUMBER SHEET: C-37


ALMADEN EXPRESSWAY
(NORTHBOUND)
100 YR. FLOW
PROPOSED HEADWALL ADDITION
Existing Headwall
PROPOSED 10' x 12' RCB ADDITION
12'
Existing Double 10' x 12' RCB
10'
10'
10'

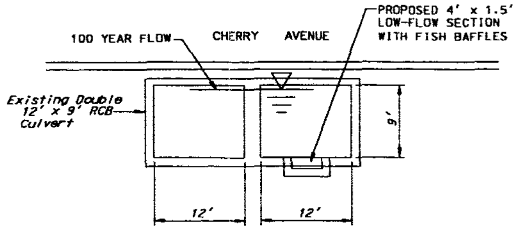
N. ALMADEN EXPRESSWAY (CANOAS CREEK)
STA.0+30± TO STA.1+40±
NTS


NIGHTINGALE DRIVE
100 YEAR FLOW
Existing Headwall
PROPOSED HEADWALL EXTENSION
9.5'
PROPOSED 9' x 9.5' RCB ADDITION
Existing Double 9' x 9.5' RCB
9'
9'
9'

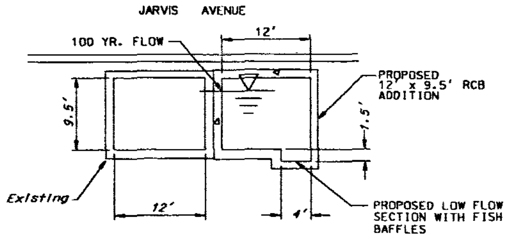
NIGHTINGALE DRIVE (CANOAS CREEK)
STA. 14+10± TO STA. 14+90±
NTS


ALMADEN EXPRESSWAY
12'
20'
9.5'
1.5'
PROPOSED LOW-FLOW SECTION WITH FISH BAFFLES
Existing
4'
100 YR. FLOW
PROPOSED 20' x 9.5' RCB ADDITION

ALMADEN EXPRESSWAY (ROSS CREEK)
STA. 2+20± TO STA. 4+30±
NTS


100 YEAR FLOW
CHERRY AVENUE
PROPOSED 4' x 1.5' LOW-FLOW SECTION WITH FISH BAFFLES
Existing Double 12' x 9' RCB Culvert
9'
12'
12'

CHERRY AVENUE (ROSS CREEK)
STA. 28+55± TO STA. 29+45±
NTS


JARVIS AVENUE
12'
100 YR. FLOW
PROPOSED 12' x 9.5' RCB ADDITION
9.5'
1.5'
Existing
12'
4'
PROPOSED LOW FLOW SECTION WITH FISH BAFFLES

JARVIS AVENUE (ROSS CREEK)
STA. 44+20± TO STA. 44+70±
NTS

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

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SANTA CLARA COUNTY CALIFORNIA
UPPER GUADALUPE RIVER
FLOOD CONTROL PROJECT
FEASIBILITY DESIGN
BRIDGES AND CULVERTS
CANOAS CREEK AND ROSS CREEK

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING BRANCH
APPROVED:
CHIEF, PLANNING/ENGINEERING DIVISION
DATE:

PREPARED UNDER THE DIRECTION OF
MICHAEL J. WALSH
LT. COLONEL C.E., DISTRICT ENGINEER

SCALE: NONE
JOB NO.:
DRAWING NUMBER
SHEET: C-38